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Subsurface drainage with small perforated flexible tubes in mole drains

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SUBSURFACE DRAINAGE WITH SMALL PERFORATED FLEXIBLE TUBES IN MOLE DRAINS

by

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INTRODUCTION

There are many areas in Iowa and other states which have subsurface drainage problems. The high cost of tile drainage in much of this land makes this practice questionable from an economical standpoint. In some areas soils have become increasingly difficult to drain because of gradual loss of topsoil by erosion or because of other factors. Under these conditions subsurface drains must be installed at narrower spacings which increase drainage cost. A method of drainage which is more economical than tile drainage is needed.

In many parts of Iowa hillside seeps are common. These are generally caused by upward movement of ground water or by seepage along impermeable layers which outcrop at or near the surface. Surface drainage normally is not effective in draining these areas. Hillside seeps frequently do not cover large areas, but their presence in cultivated fields hinders farm operations. Subsurface drainage of small isolated seepy areas may require a drain of considerable length in order to secure a satisfactory outlet. In some sections of the state the value of the land is relatively low and the drainage of isolated seepy areas with tile drains may not be economically feasible.

Natural waterways or draws in cultivated fields are sometimes wet because of seeps or because of poor natural drainage. These may be found in most parts of the state. Subsurface drains on both sides of the waterway are frequently used. Long lateral drains may also be required under these conditions. Since these drains generally run up and downhill at considerable slope, 4- and 5-inch tile drains may have greater capacity than required. There is also danger of blowouts on slopes greater than 2 or 3 per cent and bell and spigot tile are sometimes recommended.

In the southeastern part of Iowa there are relatively flat areas with impermeable subsoils at shallow depths. Placing drains at depths greater than the impermeable layer is considered undesirable because drainage is not increased and greater spacings are not desirable. The required capacity for tile drains is based on the area drained; therefore, at close spacing smaller drains can be used. Tile drains 4 or 5 inches in diameter have greater capacity than required providing the laterals are short and the slope is adequate.

Mole drainage, which is more economical in first cost than tile drainage, was investigated in Iowa by Schwab (45) and Gattis (13) and has been practiced in several other states. It has not been very successful and at best is considered a temporary means of drainage. Various investigators have attempted to stabilize mole drains, but as will be pointed out in the review of literature, none of the methods was entirely practical under field conditions.

In 1947 it was learned that thermoplastic (polyethylene) tubing was being produced commercially and that this tubing possessed physical and chemical properties which indicated that it would be suitable for mole drainage stabilization. With the cooperation of the Carter Products Corporation, Cleveland, Ohio, field investigations were started in 1948.

The Problem

The problem connected with this investigation was to find an economical method of subsurface drainage for use in areas where tile drainage is not economical or where its use is questionable. Areas with hillside seeps and soils with shallow impermeable subsoils which were previously described are examples. It is not the intent of the investigation to find a substitute for tile drains in soils where a spacing of 100 feet and a depth of 4 feet is economically feasible and adequate drainage is secured.

The use of flexible polyethylene tubing appeared to be the most promising solution to the problem. This study was limited to small-size tubes with relatively thin walls so that the cost of drainage could be held to a reasonable figure. The investigation is presented in three parts; namely, the effect of perforations on flow into subsurface drain tubes, the effect of deviations from true grade in small-size drain tubes, and the effect of wall thickness on the stability of various size tubes.

Since the drains considered here were continuous tubes impermeable to water, it was necessary to provide some means for entry of water into the drain. The problem was then to perforate the tubes with a sufficient number of holes of the proper size and shape so that inflow into the drain would not be seriously reduced because of the perforations. The flow into subsurface drain tubes in mole channels should be considered from two aspects, first as the flow into a mole channel and second as the flow into a drain through homogeneous soil which completely surrounds the drain tube. Schwab (45), Hudson and Hopewell (15), and other investigators have described the flow into mole drains as taking place primarily through the fractured soil a short distance from each side of the mole slit and vertically along the mole slit. The water moves into the drains in this manner soon after installation as the drain tubes do not then completely fill the mole channel. Under these conditions the number and size of perforations have little effect, if any, on the reduction of flow as long as there are a few perforations to permit water to enter the drain. After failure of the mole drain and consolidation of the fractured soil, the soil eventually approaches its original condition and completely surrounds the tubing. It is under these conditions that the effect of perforations on the flow into subsurface drain tubes will be evaluated.

Small-size tubes present more serious problems with regard to clogging from silt accumulation than do tile drains. For this reason the effect of deviations from true grade is considered. A large number of deviations from true grade may cause increased friction or produce surface tension effects which cause difficulties not found in drains installed on accurate grades. Drains 4 or 5 inches in diameter have sufficient cross-sectional area so that reasonable deviations from true grade do not seriously reduce their capacity. As the accuracy to which tubes are to be installed affects the design of equipment and the cost of installation, it is desirable to have such information.

Another problem of importance in the use of flexible thin-wall tubes was the stability of drain tubes in mole channels. It was believed that different wall thicknesses would be required for different size tubes. Since tubing cost varies with the quantity of material in the tube, it is desirable to keep the wall thickness to a minimum. The stability and useful life of such tubes is desired in order to compare the cost of drainage with plastic tubes to that for tile drains on an annual basis.

This investigation covers only some of the problems connected with the use of plastic tubes for drainage. Methods of installation, durability of polyethylene in soil, methods of handling and transporting tubing, and other problems were not studied. The investigation was started in 1948 and the data reported include that taken in the spring of 1951.

REVIEW OF LITERATURE

The benefits of drainage are well known and have long been recognized. The 16th Census of United States (51) taken in 1940 gives the total land in drainage enterprises in the United States as 86,967,039 acres. For Iowa this figure is 6,164,344 acres which represents over 1/6 of the total land area of the state. These acreages do not include privately-owned drainage areas of less than 500 acres. If these were included, the total land subject to drainage improvements would be considerably greater.

A report by the committee on drainage of the American Society of Agricultural Engineers by Sutton and others (49) gives some recent data as to needs for drainage in the United States. Of the land in organized drainage enterprises, 29 million acres need improved drainage, 20 million more acres can be developed by new community drains, and 8 million acres of irrigated land need drainage. This is a total of 57 million acres of land in need of better drainage.

Mole Drainage

Since the present investigation makes use of mole drains for the installation of perforated drain tubes, a brief summary of the literature on mole drainage is included. Gattis (13) and Schwab (45) compiled reviews of literature on this subject including history, limitations, and use of mole drainage. Mole drainage is considered by most authorities to be temporary in nature. During World Wars I and II when labor and materials were scarce, mole drainage was used quite extensively in many countries.

The life of mole drains was found to depend largely on the inherent stability of the soil. Childs (5) in England developed a soil moisture characteristic curve for measuring stability which was found to give a high correlation with the suitability of a soil for moling. Gattis (13) found that for three soils in Iowa mechanical analysis alone is not a good indicator of mole channel stability, but that Child's work was valid.

In the United States during the past 50 years mole drainage has been reported in New York, Iowa, Illinois, Wisconsin, California, Arkansas, Florida, Louisiana, Pennsylvania, Michigan, and Nebraska. In practically all cases mole drains have failed within a short period of time and the practice has not been generally accepted. Saveson (43) in Louisiana, and Clayton and Jones (7) in Florida reported favorably on the success of mole drains in these states.

In Great Britain mole drainage has been practiced quite extensively with success, apparently because of favorable soil and climatic conditions. Nicholson (33) found the average life of mole drains on 80 farms in England varied from 8 to 19 years. Nicholson (34) stated that mole drains act very efficiently the first 2 or 3 years and then steadily deteriorate. Under favorable conditions mole drains gave excellent results for 6 to 7 years and then new drains needed to be installed. In New Zealand mole drainage is widely practiced and extensive studies have been conducted on the installation and use of these drains. Hudson and Hopewell (15) stated that for New Zealand there was no cheaper nor more efficient method of drainage, for soils suited to it, than mole drainage. Under favorable conditions mole drains were effective for at least 10 years.

Structural Lining for Mole Drains

As a result of the inability of soil to maintain the shape of the original mole drain, several investigators have studied the problem of providing a structural lining for the mole channel. Wallem (55) reported the use of a mole plow to install tile thus providing a lining for the channel. This practice is still being utilized in many areas. A problem which arose from this method of installation was the uncertainty regarding the proper condition of the tile line, which may be caused by a broken tile or improper alignment.

Metal drain tubes

German investigators prior to World War II did considerable work on the development of machines and methods to install continuous linings in mole drains. Sack (41) developed a machine for installing a continuous metal tube by utilizing a flat strip of sheet metal. Sheet metal varying in thickness from 0.5 to 1.0 mm.(approximately 24 to 18 gauge Brown and Sharpe standard) with widths of 120 and 160 mm.(4.7 and 6.3 inches) was handled. This machine was also adaptable for installing cable or preformed tubing. The machine shown in Fig. 1 consisted of a mole plow (I) and a tube forming mechanism (II). The latter was placed against the soil bank at the beginning of the drain line and by means of properly shaped rollers (c) the flat metal strip was formed into a continuous oval tube shown in Fig. 2. The metal strips were rolled on drums (A) to facilitate transporting and to permit unrolling when the drain was installed. After the drain was pulled in, the metal tube was released by means of handle (h) on the mole plow. To close the drain a special chuck remained in the tubing at the end. The longitudinal edges of the sheet metal strip were not sealed thereby providing an opening or a slit through which water enters the drain. The sheet metal strip was generally perforated to permit the water to enter the drain more freely. A hydraulic cylinder (i) on the mole plow in Fig. 1 produced vertical movement of the mole blade maintaining the drain on grade as the mole plow moved over uneven ground.

Sack (41) also developed an instrument, which produced a beam of light, to establish the desired grade line and a

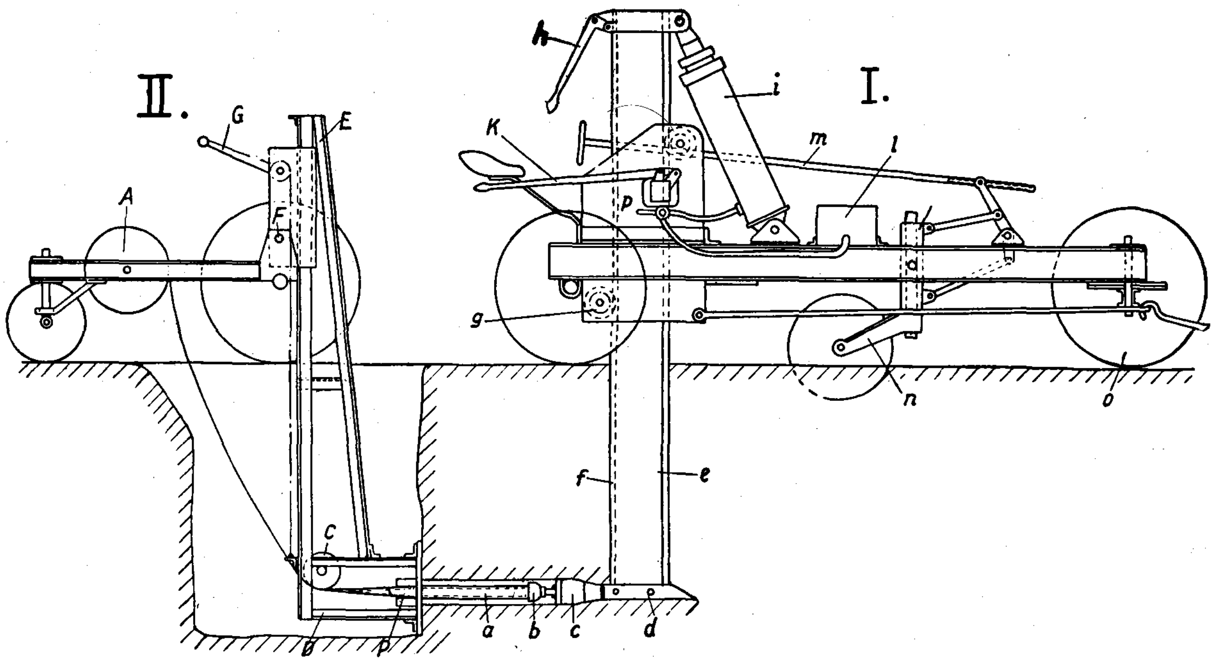


Fig. 1 Schematic Drawing of Mole Plow and Tube Forming Mechanism. From Sack (41, p. 6)

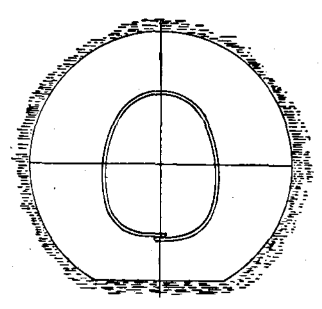


Fig. 2 Cross Section of Metal Drain Tube and Mole Drain. From Sack (41, p. 6)

mechanism on the mole plow to receive the light beam. The instrument shown in Fig. 3 consisted of a tube in which a light source (2) was mounted. The beam of light was projected toward the mole plow. In order to establish the desired grade a level bubble (4) and a telescope (3) were attached to the tube. The instrument was set up at the outlet of the drain line as shown in Fig. 4. A frosted screen was mounted on the mole blade a distance a above the drain. The beam of light was observed on the screen and the operator controlled the position of the mole blade so that the light beam intersected the horizontal cross hair. Under bright daylight conditions the light beam could be seen at a distance of about 600 feet. Sack's investigations were carried out in considerable detail including field installations. The metal for the tubes was corrosion resistant and was covered with a high grade of Japanese varnish.

Concrete drain tubes

A mole plow with special equipment was devised by Janert (17) for laying a continuous porous concrete lining in a mole channel. Sand and cement were dry mixed in the machine and the mixture moved by gravity to the mole drain as the plow moved through the soil. The dry mixture was then moistened with a small quantity of water which flowed to the mole plug and out through a porous ring of artificial pumice. The permeability of the concrete was controlled to

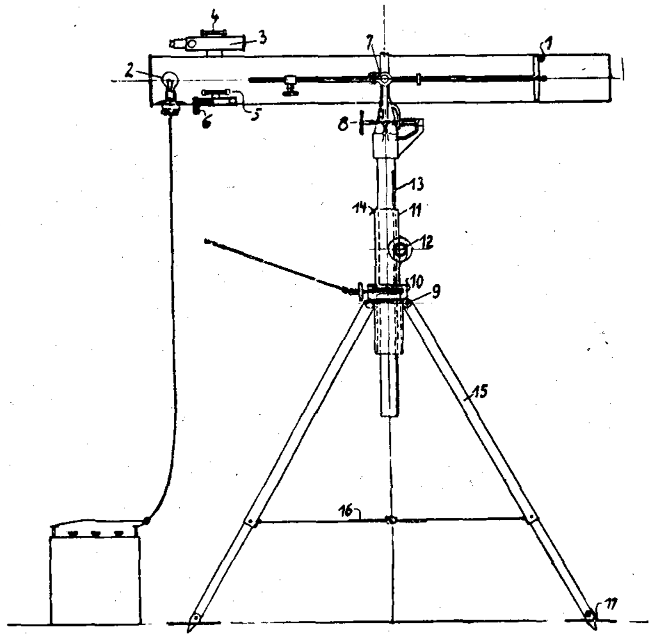


Fig. 3 Instrument for Establishing a Grade Line for a Mole Plow. From Sack (41, p. 11)

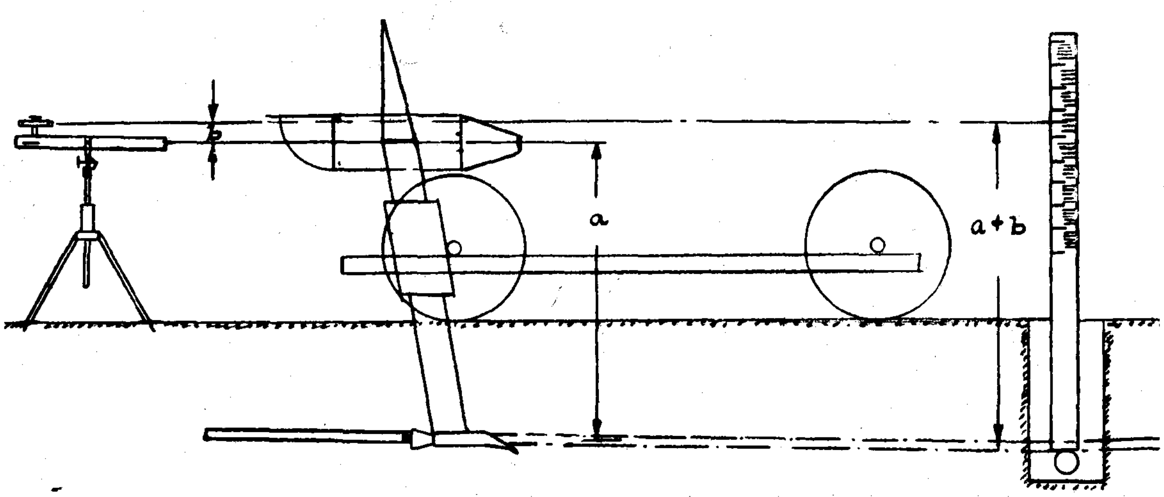


Fig. 4 Schematic Drawing Showing Use of Sighting Instrument for Establishing Grade. From Sack (41, p. 22)

a great extent by varying the proportions of cement and sand in the mixture. Janert (18) stated that admixtures and the use of graded sand increased the porosity of the pipe. A schematic drawing and a picture of the machine in the field are shown in Fig. 5. The concrete drain tube produced with such a machine is presented in Fig. 6. Sufficient water was added to the mixture to provide stability to the channel. Additional moisture was absorbed from the surrounding soil to cause the concrete to set hard.

Hopfen (14) described a later model of Janert's machine. This machine shown in Fig. 7 was mounted on a track-type tractor and installed drains to a depth of 750 mm.(29.5 inches). Hopfen (14) stated that trials were made at Leipzig and the results appeared to fulfill every expectation. U. S. Corps of Engineers (53) in 1947 requested the U. S. Army Occupation Forces to obtain further information on this method. In an interview with Dr. Dencker, professor of agriculture at the University of Bonn, Germany, he stated that results of field experiments proved that in wet soil the concrete would not dry and surface loads would cause the drain to cave in. Field experiments conducted at a later date proved that the system could be used in sand or dry terrain for the installation of subsurface irrigation systems. Janert (18) indicated that his machine was more suitable for irrigation than for drainage systems. He stated that irrigation water must be clean to prevent

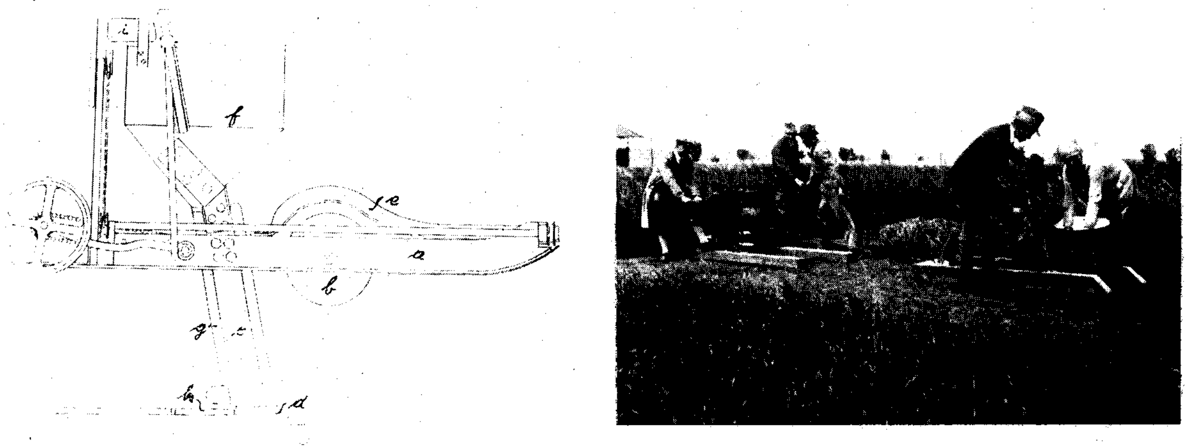


Fig. 5 Early Model of a Machine for Placing Continuous Concrete Pipe in the Soil. From Janert (17, p.284)

Fig. 6 Concrete Pipe Made with Machine Shown in Fig. 5. From Janert (17, p.284)

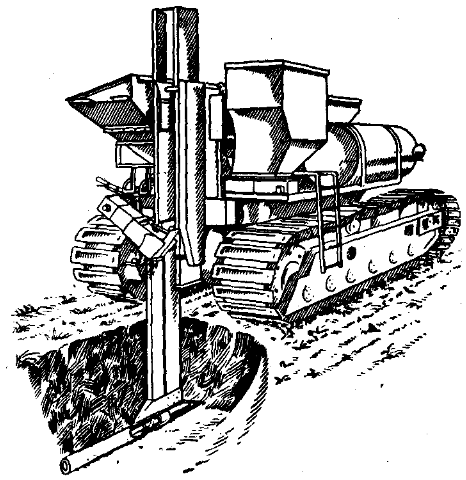


Fig. 7 Later Model of Janert's Machine for Placing Concrete Pipe. From Hopfen (71, p. 315T)

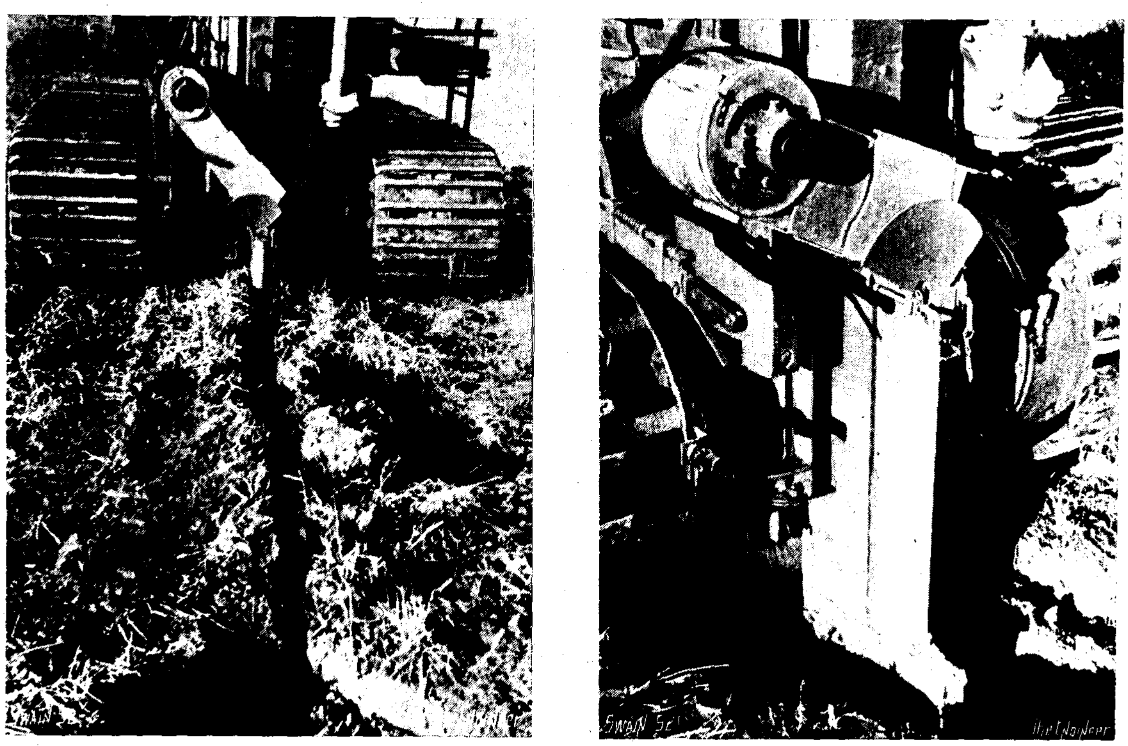


Fig. 8 Rear Views of ATG-Tubator Which is Similar to Machine Shown in Fig. 7. From (29, p. 654)

clogging, but that when the pipes were used for drainage, water from the soil did not clog the pores if the soil was in contact with the pipe so that capillary movement took place.

A machine (29) was developed in 1938 by the Allgemeine Transportanlagen A. G. of Leipzig, Germany, for making concrete pipe and was designated by the trade name ATG-Tubator Model RB. Sand, cement, and water were mixed with a screw-type mixer mounted on a track-type tractor. Mortar moved through a vertical chute to the molding screw located at the bottom of the mole blade. The molding screw was driven by a vertical shaft located in the mole blade. A specially-shaped conical mole plug centered at the rear of the molding screw shaped the inside of the pipe. A small sheet-metal shield was provided at the rear of the mole blade to prevent the concrete from being pushed up through the mole slit. The rotating molding screw forced the mortar against the sides of the mole channel pressing out any excess moisture which was in the concrete. Tubes were made with a maximum diameter of 2-1/2 inches without caving. Close-up views of the mole blade assembly of the machine are shown in Fig. 8. Manufacturer's trade literature and publications by Janert indicated that this machine was a commercial product based on his earlier models. The author was unable to verify this information. The two machines were similar in appearance and operation except that water was added at different times in the mixing process. As the manufacturer is now located in the Russian Zone of Germany, further information on this machine is not available. Janert (19) described a machine for making concrete pipes and although he did not mention the name he apparently referred to the ATG-Tubator. He stated that the machine installed tubes at the rate of 12 feet per minute at a cost of 1/10 to 1/5 that for tile drains.

The U. S. Corps of Engineers (53) conducted laboratory experiments for one year (1946-47) to determine the feasibility of draining airfields with various types of structural linings for mole drains. A large laboratory tank was filled with soil and various linings were tried. Asphalt and asphalt-sand mixtures were used and found to be impractical. Portland cement mixtures were utilized, but were not practical by pressure grouting or with a cement gun. Considerable difficulty was encountered in controlling setting time, in maintaining uniform flow, and in obtaining a continuous drain. Direct worm feed which could be synchronized with plowing speed was believed to be a possible method of extruding a Portland cement mixture. However, this method was not investigated. Janert (18) and trade literature on the ATG-Tubator showed that field installations of continuous concrete tubes had been made for subsurface irrigation. Considerable research work was done by Janert (18) on distribution of water in soil from continuous concrete pipe and he stated that these were more satisfactory than short lengths of pipe placed end to end.

Polyethylene tubes

The U. S. Corps of Engineers (53) proposed the use of 2-inch diameter perforated polyethylene tubes for airport drainage. It was proposed that the tubing be fed from a reel down a vertical blade and out the rear of a cable-laying machine. This machine was to provide a trench 3-1/2 inches wide above the drain in which a sand backfill was to be placed. A truck equipped with a hopper was to follow the machine and place the backfill material. The U. S. Corps of Engineers (53, p.37) concluded that

> the placement of a perforated plastic tubing by a cable laying machine appears to be the most promising and economical method of strengthening the walls of a mole drain.

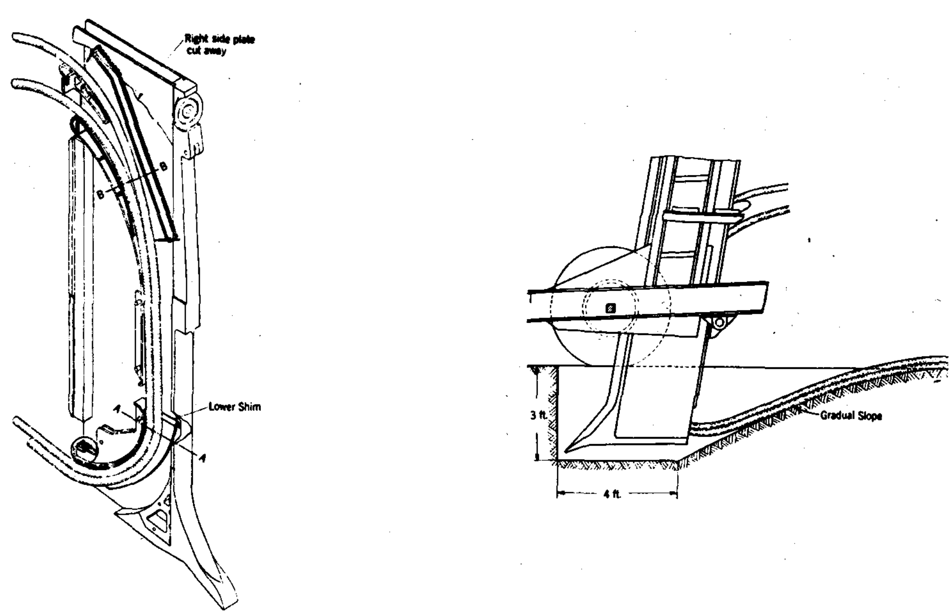
As a result of their work, they recommended the installation of perforated polyethylene tubing under field conditions to determine its effectiveness and the feasibility of the proposed method of placing. Allen (1) stated that because of lack of funds the project was discontinued and no field installations were made.

U. S. Corps of Engineers (53) described a cable-laying machine shown in Figs. 9 and 10 which was developed and used by the Wood Electrical Construction Company for installing runway lights at Standiford Field, Louisville, Kentucky. The cable-laying plows used by the American Telephone and Telegraph Company, as shown in Fig. 11, may be more suitable

Fig. 9 Front View of Cable-Laying Plow. From U. S. Corps of Engineers (53, plate 6)

Fig. 10 Rear View, Plow Turned Upside Down, of Fig. 9. From U. S. Corps of Engineers (53, plate 7)

C-60



CUT-AWAY VIEW
OF PLOW SHARE

PLOW SHARE IN
STARTING PIT

C-48

Fig. 11 Cable-Laying Plows Used by American Telephone and Telegraph Company. From U. S. Corps of Engineers (53, plates 10 and 11) as these plows accommodate larger diameter tubing at greater depths and the depth of installation can be regulated by hydraulic controls. The C-48 plow was designed to lay one or two cables with a maximum outside diameter of 2-1/2 inches to a depth of 4 feet. The C-60 plow will install cable to a depth of 5 feet.

Polyethylene was patented in the United States by Fawcett, Gibson, and Perrin (10) in April 1939 as patent number 2,153,553. Delmonte (8) stated that it was largely a British development and was first made available for manufacture in the United States in 1943. Carbide and Carbon Chemical Corp. and E. I. du Pont de Nemours and Co. now manufacture polyethylene resin. <u>Modern Plastics Encyclopedia</u> (28) reported that in England polyethylene is known as polythene or as "Alkathene", a trade name used by Imperial Chemical Industries Ltd. In the United States it is identified by its generic name, polythene, or by its chemical name, polyethylene.

Polyethylene was described by Delmonte (8, p. 72) as

> a tough, flexible polymer prepared direct from ethylene gas through a high-temperature, high-pressure polymerization process in the presence of a trace of oxygen.

He stated that it has the following chemical composition:

```
  H   H   H   H   H   H   H
- C - C - C - C - C - C - C -
  H   H   H   H   H   H   H
```

Myers (32) listed many uses which could be made of polyethylene and stated that it can be manufactured to meet the requirements of the product. Properties of resins designated as D-40 to D-145 were given. He pointed out that tensile strength may be increased by cold working and orientation of the crystals in the resin. Delmonte (8) stated that it was resistant to most chemicals at room temperature, but can be dissolved in boiling xylene and hot aromatics such as benzine. He reported that it is best known for chemical inertness and splendid dielectric properties at high frequency. Allen (1) stated that accelerated leaching tests on polyethylene tubes were conducted for 12 cycles using 5 and 15 per cent solutions of hydrochloric, sulphuric, and nitric acids. At the end of the tests the samples showed no loss in weight and visual examination indicated no harmful effects due to the acids. Some of the most important properties of polyethylene are listed in Table 1.

Flow into Subsurface Drains

Since much has been written on the flow of water through soil, this subject will be covered very briefly and only a few of the many references listed. Darcy's law states that the quantity of water flowing through a unit cross section of porous media is equal to the product of the hydraulic gradient and the permeability. Theoretical

Table 1

Properties of Polyethylene

	Delmonte (8)	Carlon Products Corp.*	Myers** (32)
Melting point, °C	100	--	--
Softening temperature, °C	--	--	108-112
Brittle temperature, °C	--	--	Below -70
Modulus of elasticity (tension), psi	--	1.5×10^5	--
Molecular weight, avg.	--	--	18-20,000
Specific gravity	0.92 (20° C)	0.95-0.98	0.92
Impact strength (ft. lb./in. of notch)	--	>3	>3
Tear strength, psi	--	--	500
Flexural strength, psi***	1700	1500-1700	--
Compressive strength, psi	--	--	3000
Tensile strength, psi (varies with temperature)	1700	1400 (120° F)	1825
Elongation, %	--	100-200	560 (ultimate at 25° C)
Thermal expansion per °C	--	18×10^{-5}	--
Sunlight resistance	--	Excellent	--
Water absorption at 25° C (% wt. gain, 24 hr.)	--	Negligible	0.01
Burning rate	--	Slow	Slow

*Trade name of product, "Carlon E and EF"
**Electrical grade D-55 (DYNH)
***See ASTM (2)

equations based on Laplace's equation and Darcy's law have been developed by Kirkham (20 and 21) for the flow of water into tile drains. To solve these equations the permeability of the soil must be known. Frevert (11), Luthin and Kirkham (25), van Bavel and Kirkham (54), and others have proposed methods for measuring soil permeability. Frevert (11) described in considerable detail the various methods in use prior to his investigation in 1948.

The effect of the width of crack between individual tile drains has been determined theoretically by Kirkham (21). Dutz (9) verified Kirkham's results using a three-dimensional electric analogue. The data checked very closely with the theoretical values provided the depth to the impervious layer and the spacing between tile lines were large. At other depths and spacings the differences between experimental and theoretical values, due to limitations in the equipment, were as large as 22 per cent. Except for the shape and distribution of the openings into the drain, the problem of the effect of crack spacing is similar to the effect of perforations. The Wheatstone bridge, electronic voltmeter, and some of the drain tube models employed by Dutz (9) were used in this work.

Electrical analogue

The flow of water by gravity through saturated soil as governed by Darcy's law is analogous to the flow of electricity through a conductor as expressed by Ohm's law. Muskat (31) pointed out that in the comparison of the two laws, the hydraulic head corresponds to the voltage, the permeability to the specific conductivity, and the rate of flow of water to the rate of current flow. He showed the relationship which exists among several problems in physics; namely, fluid flow through porous media, heat flow, current flow through a conductor, and electrostatics. The analogy of current flow to fluid flow is applicable only when the porous media, soil, is completely saturated. The electric analogue provides an excellent procedure for checking complicated mathematical solutions to ground water flow problems, as it is under saturated conditions that maximum flow takes place.

Many investigators have used the electrical analogue to solve two-dimensional problems as well as those in three dimensions. Pavlovsky (35) in 1920 first proposed the use of the electrical analogue for the solution of three-dimensional problems involving the flow of water under dams. Frevert (11) made a detailed review of literature concerning the use of the electric analogue in the solution of three-dimensional flow problems. The electric analogue has been used by Childs (6), Dutz (9), Frevert (11), Luthin and Kirkham (25), Muskat (31), Reltov (38), van Bavel and Kirkham (54), Wyckoff and Reed (57), and others in the solution of problems involving the flow of fluids through porous media.

<u>Effect of perforations</u>

When subsurface drains are made of metal pipe, fiber tar-impregnated pipe, thermoplastic tubing, and similar materials, it is generally more practical and convenient to drill or punch circular perforations in these drains than to provide other means for the entry of water. The metal casings for wells are sometimes perforated in this manner. Muskat (30) determined the effect of casing perforations on well productivity. Analytical calculations were carried out for 1/4- and 1/2-inch diameter perforations and for 6- and 12-inch diameter wells. His numerical calculations showed that the resultant well productivity was essentially independent of the perforation pattern, but was determined mainly by the total number of perforations, regardless of the detailed manner in which the perforations were distributed over the casing surface. The analytical theory was also developed for slotted perforations. The analytical approach described by Muskat (30) for the case of inflow into wells served as a basis for the analytical solution of the flow into perforated tubes as described in this work. Some of Muskat's equations were used in making the calculations as will be shown later.

Various types of perforated tubing are utilized for the drainage of highways, earth dams, bridge approaches, and other earth embankments. For such purposes engineers frequently recommend the use of well-graded sand and gravel for backfilling the trench. Although the effect of perforations in this investigation are evaluated for a tube surrounded with soil, it is of interest to present some prevailing practices. Robertson (39), drainage engineer for the Armco Drainage and Metal Products, Inc., stated that standard perforations in corrugated metal pipe (trade name, HEL-COR) 6 inches in diameter consisted of 20 5/16-inch diameter holes per foot. The holes were uniformly distributed along 4 rows with 2 rows of holes located from 30 to 60 degrees from the bottom on each side of the drain. The 2 rows of holes were spaced 1 inch from center to center along the circumference. For 12-, 15-, and 18-inch diameter pipe 3 rows of 5/16-inch holes were placed on each side of the pipe. Contrary to the above practice (holes on the bottom) the U. S. Corps of Engineers (53) proposed the use of perforated tubing by placing the perforations on the top. This was recommended in order to obtain greater inflow under saturated conditions.

U. S. Corps of Engineers (52) investigated perforated clay, concrete, and metal pipe embedded in a filter material of sand and gravel. Using full-scale models, 6-inch diameter drains were tested to determine the amount of sand and gravel washed in through various size perforations. Perforated pipe with 3/8- and 5/8-inch holes gave less wash-in than unsealed bell and spigot joints. There was also less wash-in when the holes were on the bottom of the drain than when on top. Dutz (9) in a field experiment on the effect of crack spacing between individual tile found that gravel backfill over tile with 1/8-inch crack spacing was effective in preventing movement of soil into the drains. A 1/8-inch crack spacing without gravel backfill resulted in considerable piping and a 3/8-inch crack spacing produced excessive piping. In the above experiments the subsoil contained about 20 to 60 per cent sand.

U. S. Corps of Engineers (52) observed that, in general, perforations did not limit the efficiency of the filter when the permeability was in the range of 0.1 to 0.2 cm. per second (283 to 566 ft. per day). The above tests were made using 6-inch diameter drains with perforations varying from 24 3/8-inch holes per foot to 30 5/8-inch holes per foot. Inflow into drains was greater when the holes were on the top than when on the bottom. Corrugated metal pipe with 3/8-inch diameter perforations dipped in tar resulted in a reduction of approximately 50 per cent in the effective size of the holes.

Sack (41) in his investigations with metal drain tubes utilized perforations to provide an entry for water into the drain. He suggested several distribution patterns for placing the holes in the flat metal strips. For drain tubes approximately 1-1/2 and 2 inches in diameter the number of perforations varied from 40 to 377 per meter (about 12 to

115 holes per foot) and the diameter of perforations varied from 1-1/2 to 6 mm.(approximately 1/16 to 1/4 inch). In his experiments the most commonly used drains were perforated with 117 holes per meter (about 36 holes per foot) 2 mm. in diameter (approximately 5/64 inch). For 36 5/64-inch perforations per foot in a drain tube approximately 2 inches in diameter an inflow as high as 106 liters per hour was obtained from a drain 88 meters in length (approximately 0.31 cubic feet per foot per day) 4 months after installation. Sack (41) selected the number and size of perforations in metal tubes by comparing the total cross-sectional area of the openings per unit length of tube to the cross-sectional area of the drain tube itself. As a basis for determining the number and size of perforations, previous investigations on crack spacing for tile drains were considered. For example, he assumed a crack spacing of 1-1/2 mm. to be adequate for tile drains 5 cm. in diameter. For the same diameter metal tubes he assumed a slit width of 1/2 mm. With 40 perforations, 3 mm. in diameter, per meter approximately the same cross-sectional area for the openings was obtained for the metal tubes as for tile drains. For drains 5 cm. in diameter in peat soil he used 98 perforations, 6 mm. in diameter, per meter for metal tubes which was equivalent approximately to 4 mm. crack width for tile drains. The effect of the perforations on inflow was not evaluated.

Deviation from True Grade in Small Drain Tubes

The effect of air entrapped between drops of a liquid in a horizontal capillary tube was first observed by Jamin (16) in 1860. This phenomenon, known as Jamin's tubes, was defined by Poynting and Thomson (36, p. 142) as follows:

> Jamin's tubes are capillary tubes containing a large number of detached drops of liquid; these can stand an enormous difference of pressure between the ends of the tube without any appreciable movement of the drops along the tube.

They pointed out that by observing the liquid drops in capillary tubes the curvature of the ends of each drop was different; thus each drop transmitted a smaller pressure than it received. If there were a large number of drops, the difference in pressure between the ends of the capillary tube amounted to several atmospheres.

Jamin (16) reported that a pressure at the end of the tube caused movement of the separated water drops for a distance along the tube which was proportional to the pressure applied. He stated that the pressure was proportional to the number of air and water parcels, independent of the length of each water drop, increased when the air bubbles decreased in size, and increased very rapidly when the diameter of the tube was made smaller.

Stability of Flexible Conduits

Considerable research work has been conducted on loading and strength of flexible conduits. This work was carried out for large diameter pipe placed under embankments subjecting the pipe to high pressures, such as in road fills. Spangler (48) conducted full-scale field experiments with flexible corrugated metal pipe 36 to 60 inches in diameter and developed an equation to determine the ultimate horizontal deflection of the pipe. Rahman (37) developed a theoretical deflection formula for the determination of the deflection of flexible circular rings at any point on the circumference. He assumed the pipe to be acting under two equal and opposite concentrated loads and to be stressed beyond the elastic limit. With further development and modification the formula may be applicable under field conditions. The investigations on large diameter pipe are not directly applicable to small plastic tubes; however, the type of failure is similar.

The American Society of Testing Materials (2) has developed a tentative test for determining the stiffness in flexure of flat strips of nonrigid plastics. Further investigations are necessary to determine the relation of the above methods and procedures to the stability of small flexible pipe such as polyethylene tubing.

The U. S. Corps of Engineers (53) conducted tests on

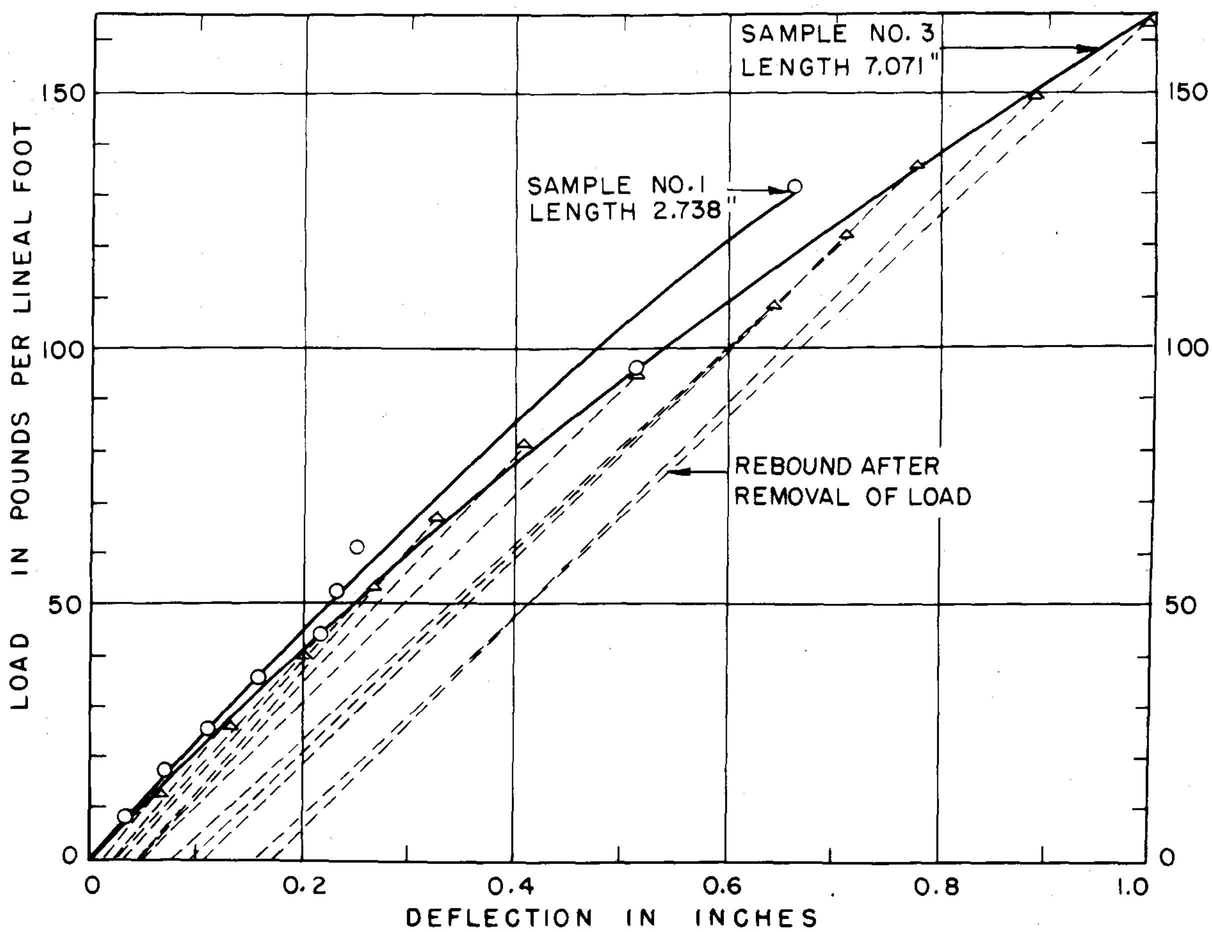


Fig. 12 Load-Deflection Curves for 2-inch Diameter Plastic Tubing. Redrawn from U. S. Corps of Engineers (53, plate 43)

short lengths of 2-inch diameter tubing to determine deformation and rebound of the tubing under alternate loading and releasing of the load. Although not stated, it is assumed from photographs and descriptions that the 2-inch tubing tested was of standard size and wall thickness as indicated in Table 17. The tubing was loaded between two flat surfaces and the load was allowed to remain until there was no deflection. The load-deflection curves are shown in Fig. 12. The dashed lines show the rebound of the tubing after release of the load. With a load of 100 pounds per foot the tubing deflected approximately 0.5 inch. After removal of the load the tubing was compressed 0.1 inch.

INVESTIGATION

This investigation was conducted in three parts; namely, the effect of circular perforations on the flow into subsurface drain tubes, the effect of deviations from true grade on the performance of 1-inch drain tubes, and the effect of flexible plastic tubes in stabilizing mole drain channels under field conditions. The investigation of each of the three problems was carried out independently and is reported in a separate section of the thesis. The studies began in 1948 and were a continuation of research on mole drainage by Schwab (45) in 1947 and Gattis (13) in 1949. The results up to and including the spring of 1951 are discussed here. The ultimate objective of this investigation was to develop a practical method of subsurface drainage which was more economical than tile drainage. Perforated flexible thermoplastic tubing for stabilizing mole drains is proposed as a solution to the problem.

Effect of Circular Perforations on Flow into Subsurface Drain Tubes

A continuous drain such as a thermoplastic tube which has been proposed for subsurface drainage must be provided with openings along its length for the entry of water. In tile drains these openings are provided by the cracks between individual lengths of tile. For practical reasons these openings in continuous tubes are generally circular. The number and size of perforations for a given diameter of drain need to be known in order to provide adequate drainage consistent with economical cost of manufacture of the tubing.

Various types of perforated tubing are used for drainage in earth dams, bridge approaches, and other earth embankments. Several types of commercially available perforated tubing are shown in Fig. 13. These pipe are known as (A) corrugated metal pipe (trade name HEL-COR), (B) fiber tar-impregnated pipe (trade name Orangeburg), and (C) polyethylene tubing (trade name Carlon E).

Theory

The analytical solution* of the flow into perforated drain tubes was derived not only to show the way in which various factors enter into the problem, but also to obviate a large amount of experimental work. By comparing the flow into a perforated drain tube with that into an "open" drain (essentially a drain embedded in gravel) as computed from an equation derived by Kirkham (20), the effect of the perforations can be determined.

The derivation of subsequent equations is based on the following assumptions: (1) the soil is water-saturated to

*Dr. Don Kirkham is credited with the derivation of the theoretical analysis which follows.

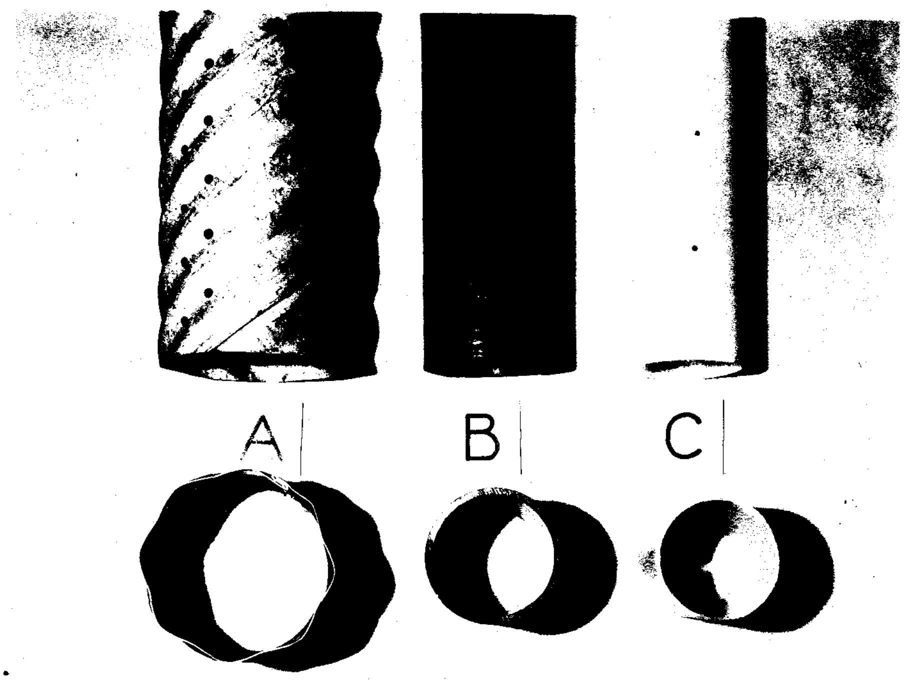


Fig. 13 Types of Perforated Tubing Used in Subsurface Drainage

the surface and in addition has a layer of ponded water of thickness <u>t</u>, (2) the drain tubes are running full with zero back pressure, (3) the permeability of the soil is the same whether measured in the horizontal or vertical direction, (4) the soil, both around the drain tube and at distances removed from it, is of uniform permeability, (5) the drains are spaced far apart compared to their depth, (6) an impervious layer is at considerable depth, preventing deep natural drainage of the soil. Item (6) will have little effect on the solutions if the drain tube is a few radii above an impervious layer. In addition to the above six items, certain simplifying assumptions in the analysis have been made. For practical purposes, however, the results may be taken as correct.

A diagrammatic sketch illustrating the problem and showing geometrical quantities involved is shown in Fig. 14. In order to simplify subsequent discussion and presentation of tables and graphs and to provide a ready reference for symbols to be used in the analysis, the following definitions are given.

a	longitudinal spacing of perforations
C	a constant defined by equation (7)
d	depth from soil surface to center of drain tube
i	an integer having values 1, 2, 3, ...(m - 1)
K	soil permeability
K_o	Bessel functions of the second kind and zero order

m	number of longitudinal rows of perforations
n	an integer having values 1, 2, 3, ...
q	a flow coefficient (constant)
Q	flow into a perforated drain tube (circular plate-shaped openings) of unit length in unit time
Q_o	flow into an "open" (gravel-embedded) drain tube of unit length in unit time
Q_s	flow into a perforated drain tube (spherical openings) of unit length in unit time
r_i	distance from center of perforation at (x_i, y_i) to point (x, y) (See Fig. 14)
r_i'	distance from center of perforation at (x_i', y_i') to point (x, y) (See Fig. 14)
r_p	radius of a perforation
r_w	radius of drain tube (measured to outside circumference)
R	resistance of electrolyte corresponding to flow, Q
R_o	resistance of electrolyte corresponding to flow, Q_o
R_s	resistance of electrolyte corresponding to flow, Q_s
t	thickness of surface water on soil
x, y, z	rectangular coordinates in X-, Y-, Z-planes, respectively

θ angle measured from X-axis to a line drawn from the center of the perforations to the origin of the coordinates (See Fig. 14)

θ_i same angle as θ except measured in the opposite direction on the "image drain" (See Fig. 14)

In following the subsequent analysis the reader should be familiar with the methods (see for example, Muskat (31)) for solving ground water problems governed by Darcy's law. Besides the actual drain tube of radius r_w* at depth d below the soil surface, there is an image drain tube at distance d (and radius r_w) above the surface as shown in Fig. 14. The purpose of the image drain is to make, analytically, the surface of the soil a plane of equal hydraulic head; for it is assumed that the soil is water-saturated to the surface, and that there can also be a layer of water of thickness t on the surface of the soil.

The perforations are of radius r_p and lie on m longitudinal lines, equally spaced about the periphery of the tube. The longitudinal separation of the perforations is a. The location of the perforations is further specified by placing them on equally spaced circles having their centers on the axis of the tube. The origin of rectangular coordinates is taken at the center of one of these circles. The Y-axis is vertically upward, the X-axis horizontally to

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*The notation in the present analysis agrees largely with that of Muskat (30), his w denoting "well".

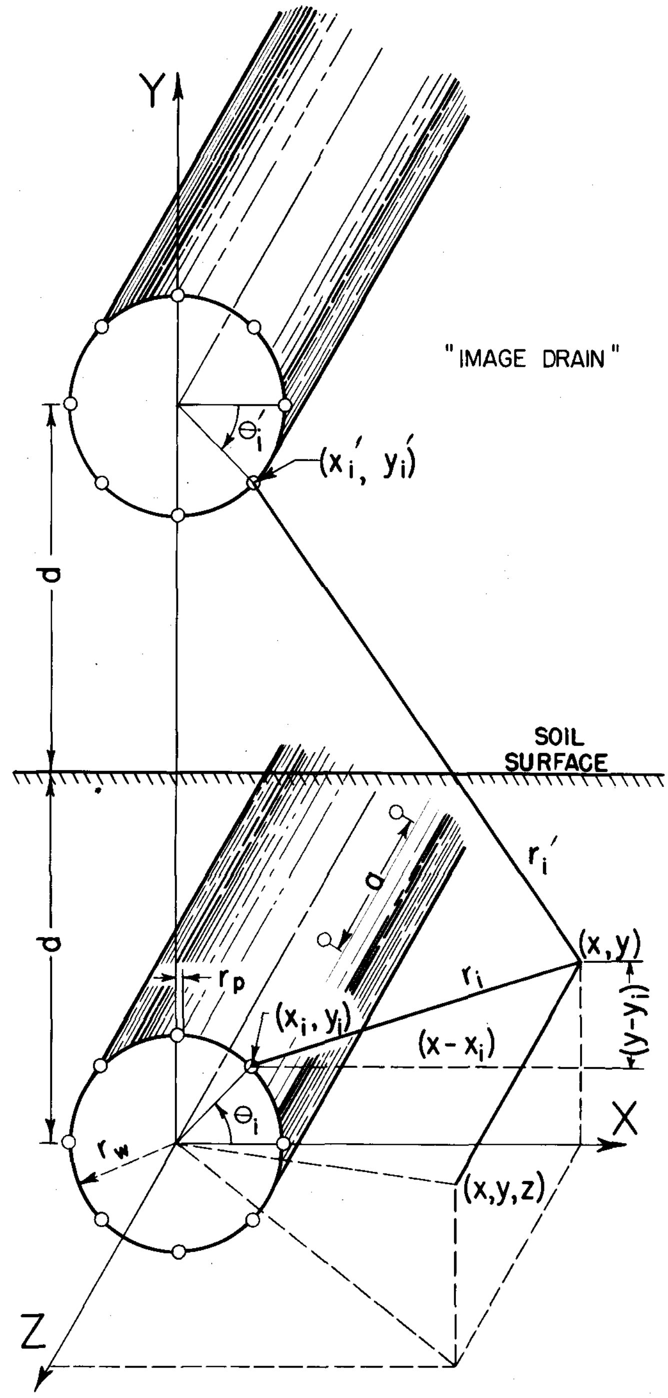

Fig. 14 Diagrammatic Representation of a Perforated Sub-surface Drain Tube and Its Image

the right, and the Z-axis is colinear with the axis of the drain tube. The location of a typical perforation lying on the circle in the (X, Y)-plane is specified by the coordinates of its center (x_i, y_i), or by the angle θ_i measured from the X-axis to a line drawn from the center of the perforations to the origin of coordinates.

Determination of the hydraulic head. An expression for the hydraulic head h (h is a potential function) at a point (x, y, z) in the soil is first obtained. With h known, the flow into the perforations can readily be computed. The analytical form desired can be taken basically as that due to a vertical perforated well casing, a form derived by Muskat (30). In the present problem the well casing becomes a horizontal drain tube; an image "well casing", as indicated above, is also required. Muskat assumes that the perforations approximate analytically a series of spherical sinks. This assumption is also used initially here. Later in the analysis it will be shown that the flow values obtained on the basis of spherical sinks must be multiplied by a factor $2/\pi = 0.636$ to yield the proper flow for flat, circular openings, as actually found.

Let the projection on the (X, Y)-plane at the point (x, y, z) (Fig. 14) be distance r_i from the center of the perforation at (x_i, y_i) and let the projection of this same point be distance r_i' from the image perforation at (x_i', y_i'). Then from the figure

$$r_i = \left[(x - x_i)^2 + (y - y_i)^2\right]^{1/2}$$
$$r_i' = \left[(x - x_i')^2 + (y - y_i')^2\right]^{1/2} \tag{1}$$

Also, it is convenient to define

$$\rho_i = r_i/a, \rho_i' = r_i'/a, \rho_p = r_p/a;$$
$$w = z/a, \ \delta = d/a, \ \rho_w = r_w/a; \tag{2}$$

and it is observed from the figure that

$$x_i = x_i' = r_w \cos \theta_i,$$
$$y_i = r_w \sin \theta_i,$$
$$y_i' = 2d - r_w \sin \theta_i,$$
$$x_0 = r_w, \ y_0 = 0. \tag{3}$$

Using the above notation one can now write down the potential due to all the perforations in two lines of perforations θ_i and θ_i'. This potential h_i is

$$h_i = -\frac{4q}{a}\left\{\log_e\left(\frac{2}{\rho_i}\right) + 2\sum_{n=1}^{\infty}\left[K_o(2n\pi\rho_i)\right]\left[\cos\ 2n\pi w\right]\right\}$$
$$+\frac{4q}{a}\left\{\log_e\left(\frac{2}{\rho_i'}\right) + 2\sum_{n=1}^{\infty}\left[K_o(2n\pi\rho_i')\right]\left[\cos\ 2n\pi w\right]\right\} \tag{4}$$

where $K_o(2n\pi\rho_i)$ and $K_o(2n\pi\rho_i')$ are Bessel functions of the second kind and zero order (sometimes also called Hankel functions), defined and tabulated, for example, in the <u>British Association Mathematical Tables</u> (4), and <u>q</u> is a constant, a flow coefficient, to be evaluated. The form of the expressions in brackets in equation (4) differs from the form of Muskat in that a subtractive term ($2n\pi\, i/m$) is not included in the argument of the cosine term. This term was used by Muskat because he assumed that the perforations lay on a spiral, whereas they are taken here to lie on equally spaced circles. Muskat showed that the results are for practical purposes the same in either case. The circle case is simpler analytically.

The occurrence of the logarithmic term $\log_e(2/\rho_1)$, and hence of $\log_e(2/\rho_1')$ in equation (4) is explained by Madelung (27)* (see especially pp. 525-527), from whom the potential form stems. Although in equation (4) the 2's in the numerators of the logarithmic terms can be cancelled, they are preserved to retain the basic potential form. Muskat's <u>q</u> differs from the present <u>q</u> by a constant factor, since he deals with pressures rather than hydraulic heads.

*Madelung's E corresponds to the 2q used by Muskat. Muskat does not state why 2q instead of the conventional q is used. The reason, however, seems to lie in the fact, brought out in the appendix of his article, that the flow into the perforations may be assumed to be not only equivalent to the flow into spherical sinks, but also to twice the flow (nearly) which would pass into hemispherical sinks located at the perforations. These hemispherical sinks are tacitly introduced to account analytically for the physical fact that the well-casing is impervious except at perforations, there being accordingly no possibility for flow to enter the halves of spheres on the inside of an impervious tube wall.

The potential h due to all the m rows of sinks is obtained from equation (4) by summing the effects of the individual pairs of rows, and by adding a term $(d + t)$ which makes $h = d + t$ when $y = d$. This potential is (since $h_i = 0$ when $y = d$)

$$h = \sum_{i=0}^{m-1} h_i + d + t \tag{5}$$

where the reference level for hydraulic head is taken at the level of the axis of the drain tube, assumed horizontal, and a layer of water of thickness t is assumed to be on the surface of the water-saturated soil. The h_i terms are zero when $y = d$, because of the use of the image drain tube.

The flow constant q is now evaluated. This is done by determining the analytical value of h (say h_w) at a reference point on a perforation and equating this value to the known physical value of h at this point of reference. The known value results from the assumption that the drain is running full with no back pressure, so that the hydraulic head (referred as before to the level of the drain axis) has the constant value r_w everywhere inside the drain tube, and hence has this value r_w at each perforation. Therefore $h_w = r_w$. It is convenient to evaluate h_w at a point on the perforation centered at $(x_i, y_i) = (x_o, y_o) = (r_w, 0)$, the coordinates of this point being $(x, y, z) = (r_w, r_p, 0)$. Substituting these values of x, y, x_i, y_i, and z in equation (5), one finds with the aid of equations (2) to (4), subject to comments below,

$$
\begin{aligned}
h_w = r_w = -\frac{4q}{a} & \left\{ \log_e \left(\frac{2}{\rho_p}\right) + 2\sum_{n=1}^{\infty} K_o(2n\pi\rho_p) \right. \\
& + 2\sum_{i=1}^{m-1}\left[\sum_{n=1}^{\infty} K_o\left(4n\pi\rho_w \sin\frac{\theta_i}{2}\right)\right] \\
& \left. -(m-1)\log_e \rho_w - \sum_{i=1}^{m-1}\log_e \sin\frac{\theta_i}{2}\right\} \qquad (6) \\
& + \frac{4mq}{a}\left\{\log_e\left[\frac{2}{(4\delta^2+\rho_w^2)^{\frac{1}{2}}}\right]\right. \\
& \left. + 2\sum_{n=1}^{\infty} K_o\left[2n\pi(4\delta^2+\rho_w^2)^{\frac{1}{2}}\right]\right\} + d + t,
\end{aligned}
$$

which may be solved for q.

In equation (6) the term with coefficient $-4q/a$ represents the effect of the perforations in the actual drain tube. This term can be obtained directly from the right-hand side of Muskat's (30) equation (6) by taking his quantity $\cos 2n\pi i/m = 1$, for the reason indicated previously. The term with coefficient $4mq/a$ in equation (6) is that due to the image drain. The latter term has been determined by assuming that an average value of ρ_1' evaluated at the reference point is $(4\delta^2 + \rho_w^2)^{1/2}$, and that, consequently the effect of the m image lines is merely m times the average value of one of them. For definiteness (as well as convenience) the perforation centered at $(r_w, 0, 0)$ has been singled out to obtain equation (6), and hence evaluate q. If, instead, the perforation at $(0, r_w, 0)$ had been used, a larger value of q would have resulted. Actually an average value of q appropriate for all the perforations is desired. Consideration shows that this average value will be given by equation (6) if ρ_w^2 is dropped compared with $4\delta^2$. An easy way to reach this conclusion is to measure, and then average, all possible distances between the perforations shown on the two large circles in Fig. 14. It will be found that the average distance is very nearly 2d, the approximation becoming better as 2d becomes large compared to r_w. For example, if there are four lines of perforations and d is as small as $d = 2r_w$, the average distance referred to is found to be 2.06d; whereas if $d = 4r_w$, the average distance is 2.02d. The apparent errors, 3 and 1 per cent, are made still smaller by the circumstance that these distances 2.06d and 2.02d occur in the arguments of slowly varying logarithmic, and Bessel function terms.

Aside from replacement of $(4\delta^2 + \rho_w^2)^{1/2}$ in equation (6) by 2δ, the equation may be further simplified by introducing a constant C defined by

$$C = \frac{1}{m}\left\{2\sum_{n=1}^{\infty} K_o(2n\pi\rho_p) + 2\sum_{i=1}^{m-1}\left[\sum_{n=1}^{\infty} K_o(4n\pi\rho_w \sin\frac{\theta_i}{2})\right] + \log_e(\frac{\rho_w}{\rho_p}) - \sum_{i=1}^{m-1}\log_e(2\sin\frac{\theta_i}{2})\right\}. \tag{7}$$

Using equation (7) in equation (6), noting that for practical cases the K_o terms involving δ are negligible and simplifying and solving for q, there results

$$q = \frac{(a/4m)\ (d + t - r_w)}{C + \log_e\ (2d/r_w)} \tag{8}$$

Computation of the flow. Since q is now known, h is completely determined and the flow Q_S per unit length of drain tube per unit time can be computed. This is done by noting that at all points at the surface of the soil the flow Q_S' per unit area per unit time is vertically downward (the subscript S referring to spherical perforations), and hence, by Darcy's law, is given by

$$Q_S' = K\ \partial h/\partial y,$$

where $\partial h/\partial y$, the hydraulic gradient, is evaluated at y = d and where K (not to be confused with the Bessel function K_o)

is the soil permeability. The total flow Q_S entering the drain tube is, therefore,

$$Q_S = K \int_{-\infty}^{\infty} (\partial h/\partial y)_{y=d}\, dx. \tag{9}$$

In evaluating $\partial h/\partial y$ one makes use of the fact that near y=d the K_o-terms in the hydraulic head function h are essentially constant (= zero) compared to the logarithmic terms; and that the derivative of K_o terms are thus essentially zero compared to the derivatives of the logarithmic terms. Therefore, near y=d, very approximately,

$$h = -\frac{4q}{a} \sum_{i=0}^{m-1} \left[\log_e \left(\frac{2}{\rho_i}\right) - \log_e \left(\frac{2}{\rho_i'}\right) \right] + d + t. \tag{10}$$

That is, near y=d, using equations (1) to (5) in equation (10) and simplifying, there results

$$h = \frac{2q}{a} \sum_{i=0}^{m-1} \Big\{ \log_e \left[(x - r_w \cos \theta_i)^2 + (y - r_w \sin \theta_i)^2 \right]$$
$$- \log_e \left[(x - r_w \cos \theta_i)^2 + (y - 2d + r_w \sin \theta_i)^2 \right] \Big\} + d + t;$$

so that

$$\left[\frac{\partial h}{\partial y}\right]_{y=d} = \frac{8q}{a} \sum_{i=0}^{m-1} \left\{ (d - r_w \sin e_i) \left[(x - r_w \cos e_i)^2 + (d - r_w \sin e_i)^2 \right]^{-1} \right\} . \qquad (11)$$

Putting equation (11) in equation (9) and integrating, there results

$$Q_S = 8Kq\pi m/a;$$

or substituting in this expression the value of <u>q</u> given by equation (8), one finds

$$Q_S = 2\pi K(d + t - r_w)/[C + \log_e (2\, d/r_w)] \qquad (12)$$

<u>Derivation of the factor $2/\pi$</u>. The value Q_S so far derived is for spherical sinks and is thus higher than the flow value one would expect to find in practice. Soil is generally not bulged up at drain tube openings. It lies flat over them, or even protrudes inwardly. To take the latter actual situation into account the reasoning is as follows: Muskat (30) showed (see the appendix of his article) that well-casing perforations, although located on an impermeable pipe, nevertheless, behave as if they were

an array of spherical sinks located in the porous medium devoid of pipe. Muskat's assumption that the shape of the sinks is spherical is justified in that the perforations of his interest are made by firing bullets into a well casing, a process which opens up porous medium about the perforations. In the present case there is little justification for assuming spherical-shaped sinks. On the contrary, as indicated above, one can conclude that circular plate-shaped sinks are appropriate to the present flow and that a comparison of flow into circular plate-shaped sinks and into spherical sinks should yield the factor sought. Since Darcy's law, Ohm's law, and Maxwell's law, are all analogous, it can be shown (compare Muskat (31) p. 140 with Smythe (47) p. 233) that the electrostatic capacities of two condensors are in the same ratio as are the seepage flows for two analogous ground-water systems. Now the electrostatic capacity for a circular plate of radius r_p in an infinite medium of dielectric constant unity is $2r_p/\pi$ (see Smythe (47) p. 112, equation (2)), whereas the capacity for a sphere of radius r_p in the same medium is merely r_p (see Smythe (47), bottom p. 27, equation (2)). The ratio of these two quantities, $(2r_p/\pi)/(r_p) = 2/\pi$, is the ratio sought if it can be shown that the spherical sinks and circular plate-shaped sinks in the actual problem are sufficiently far apart as to behave as if they were infinitely separated. The latter point may be shown as follows.

The electrostatic capacity C_E of two spheres of radius r_p and distance c between centers is given by*

$$C_E = 2r_p \sinh \beta \sum_{n=1}^{\infty} \left[\operatorname{csch} (2n-1)\beta - \operatorname{csch} 2n\beta\right]$$

where β is defined by

$$\operatorname{csch} \beta = c/2r_p.$$

Since perforations will generally be at least 5 diameters apart $c/2r_p = 5$, $\beta = 2.292$, $\sinh \beta = 4.897$, $\Sigma = 0.1857$, and $C_E = 2r_p \times 4.897 \times 0.1857 = 2r_p \times 0.910$. Thus the proximity of the spheres results in a decrease in their capacity of only 9 per cent compared with what it would be for infinite separation. An exact expression for the capacity of a pair of circular plates is not available, but the error due to proximity should be of the same order of magnitude as for spheres. It is concluded then that the factor $2/\pi$ is sufficiently accurate for practical purposes. Therefore, writing Q for the flow per unit length into a perforated drain tube having no outward (or inward) bulging of soil over the perforations, the multiplication of both sides of equation (12) by $2/\pi$ yields, since $Q = Q_s(2/\pi)$,

$$Q = 4K(d + t - r_w)/\left[C + \log_e (2d/r_w)\right]. \qquad (13)$$

*See Smythe (47) p. 120, equations (3), (4), and (5), and page 37, equation (1), taking $V_1 = V_2 = V$, and $Q_1 = Q_2 = Q/2$, so that $C = Q/V = 2(c_{11} + c_{21}) = 2(c_{12} + c_{22})$.

The flow Q_s and Q of equation (12) and (13) may be compared with the flow Q_o into a completely porous drain tube. A completely porous drain tube is approximated, in practice, by a drain embedded in gravel. This flow Q_o is found to be (see, for example, Kirkham (20), equation (11) where r_w is to be neglected compared to d)

$$Q_o = 2\pi K(d + t - r_w)/\left[\log_e (2d/r_w)\right] . \qquad (14)$$

Although equation (14) is strictly true, for an impervious layer at great depth (and for spacing of drains large compared to d), the equation is, in addition, very nearly correct for drain tubes located a few drain radii above an impervious layer. Equations (12) and (13) should likewise be true if the drain tube is a few drain radii above an impervious layer.

Apparatus and materials

Equations (12), (13), and (14) were verified using electric models to simulate seepage flow. Electric models or analogues are based on the observation that Ohm's law and Darcy's law are analogous. Thus, seepage flow corresponds to electric current; hydraulic gradient, to voltage gradient; soil permeability, to specific electrical conductivity; impermeable boundaries, to insulators; and surfaces of equal hydraulic head, to conductors at constant potential. A principal advantage of the electric analogue is that uniform conductivity for the flow medium is easily achieved. This is not the case in a field or laboratory test, in which more or less puddled soil is necessarily used. With soil it is difficult, if not impossible, to obtain uniform water permeability in a reasonable time. In the field, many years may elapse before soil in a tile trench assumes a condition approaching that existing before excavation. Sand models obviate puddling but are less flexible than electric models. Childs (6) has used two-dimensional electric analogues in the study of some agricultural drainage problems. Frevert (11) recently developed a three-dimensional electric analogue for the study of measurement of soil permeability; this apparatus in modified form has been used by Frevert and Kirkham (12), van Bavel and Kirkham (54), and by Luthin and Kirkham (25). As the present problem was a three-dimensional one, some of Frevert's apparatus has again been used in these tests.

Fig. 15 shows five electric models, all representing 2-foot lengths of drain tubes used in this study. For convenience in construction models and tank assembly were made to a scale of 2 to 1. Only 1-foot lengths (or some other convenient length) need be considered since the flow is the same into each unit length of drain tube--excluding, of course, the case that successive lengths have differing arrangements of perforations. Models 1 and 2 were designed on the assumption that the perforations are equivalent to

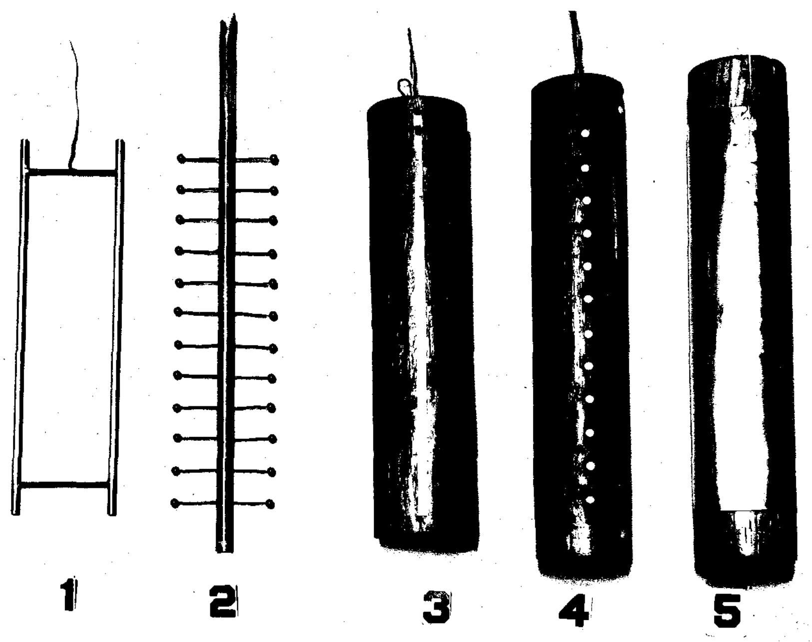

Fig. 15. Drain Tube Models Used in the Electric Analogue for Seepage Flow into Drain Tube Perforations

spherical sinks, model 1 being a special case of model 2. Model 2 consisted of two lines of spherical electrodes supported by insulated copper wires which were held in place by a lucite tube. Polished ball bearings which were used for the electrodes were soldered to 20 gauge copper wire. The wire, aside from holding the metal balls in place, served to conduct (through the center of the lucite tube) electric current from the balls to an external voltage supply. The insulated wires and the lucite tube were kept small in order to simulate more nearly the assumed condition that the size of the wires and lucite tube should be infinitesimal. The 1/2-inch diameter lucite tube was split longitudinally, the insulated lead wire and wire to the balls was inserted, and the tube then filled with hot paraffin. Model 1 represented a limiting case of model 2, the case when the distance between the centers of the spherical sinks (dimension a of Fig. 14) was zero. Model 1 consisted merely of two parallel cylindrical electrodes held apart rigidly.

Models 3 and 4 corresponded to models 1 and 2, respectively, except that circular plate-shaped electrodes replaced the spheres, and that the flat electrodes were set in a cylindrical shellaced wooden block, simulating a drain tube, rather than being suspended in space as in models 1 and 2. Model 4 was constructed by splitting a wooden cylinder along its longitudinal axis and by fitting in 3-inch lengths of bronze welding rod which were covered with glyptol except on their ends. The two halves of the model were placed together with the ends of the electrodes flush with the surface of the cylinder. The wooden cylinder was then painted with several coats of shellac. Model 3 corresponded to a drain tube slotted longitudinally on diametrally opposite sides; that is, the spacing of the circular plate-shaped electrodes was zero. This model was made with two copper strips whose width was equal to the plate diameter. Each strip was insulated with glyptol on the side next to the wooden cylinder.

Model 5 represented a completely pervious drain tube; that is, it represents, practically speaking, a drain tube embedded in gravel. For this case, since now the whole drain tube surface is at equal hydraulic head, the 1-foot long model becomes a cylindrical electrode. All the models were mounted in turn in a tank assembly as shown in Fig. 16.

The side walls of the tank assembly were dielectric materials, the walls here simulating the conceivable field condition that thin strips of impermeable material could be placed at locations two feet apart and perpendicular to a drain tube without disturbing the flow. The side walls of the tank assembly, which were made of masonite, were utilized to support the drain tube models and to insulate the water in the tank assembly from that in the stock watering tank. Several coats of shellac were painted on the masonite to prevent absorption of water. The bottom of the tank

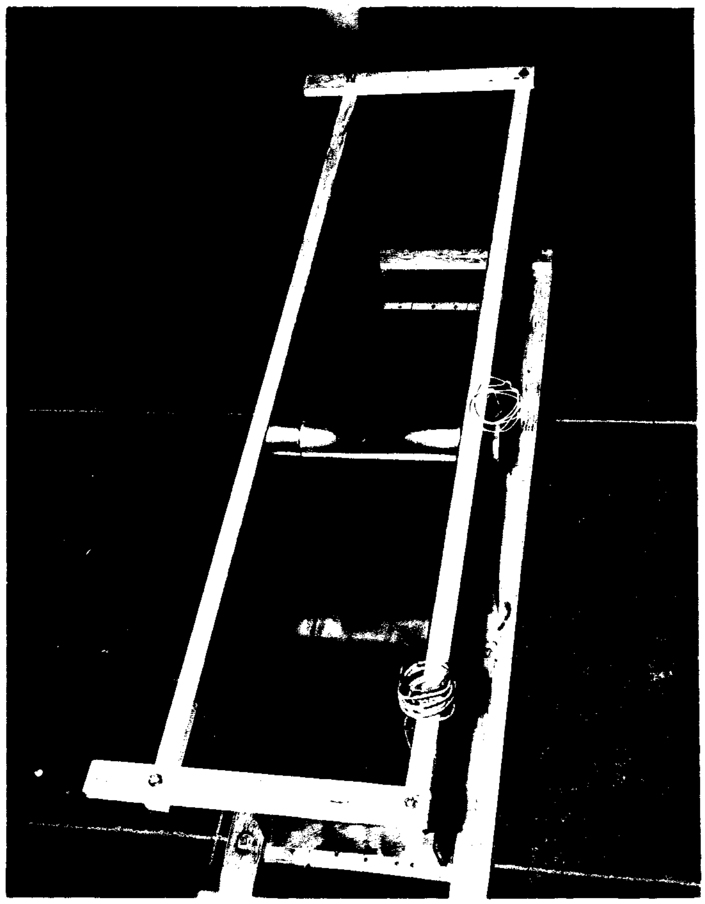

Fig. 16 Tank Assembly for Holding Drain Tube Models Shown in Fig. 15

assembly was a sheet of copper; the top and ends were open. If the top and ends were made of dielectric sheets, and if the tank assembly were filled with electrolyte and turned upside down, then the copper sheet, if held at a certain voltage, would represent the water table, the face opposite the copper sheet would correspond to an impermeable layer in the soil, and the ends of the tank would correspond to impermeable end boundaries. Since in this case, a horizontal water table was under consideration, most of the flow per unit time entered the drain from a relatively small volume of soil above the drain tube according to Kirkham (20). Therefore, what happened at the ends of the tank was immaterial provided no voltages were applied. The unit shown in Fig. 16 was placed in a wooden stock watering tank (Fig. 17) which was available from previous model work. The assembly was placed in the larger tank with the copper side down. The air above the water-filled tank now represented an impervious layer beneath the drain tube. In the electric analogue it was possible to place the model upside down because electric force, not gravity, caused the flow. The stock watering tank with tank assembly and external electrical equipment is shown in operating position in Fig. 17. The equipment consisted of a 6-volt 60-cycle transformer (1), a Wheatstone bridge (2), an electronic voltmeter (3), and tank assembly shown in Fig. 16. Tap water served as electrolyte.

Fig. 17 Stock Water Tank, Tank Assembly and Drain Tube, and External Electrical Equipment for Model Tests of Theoretical Equations

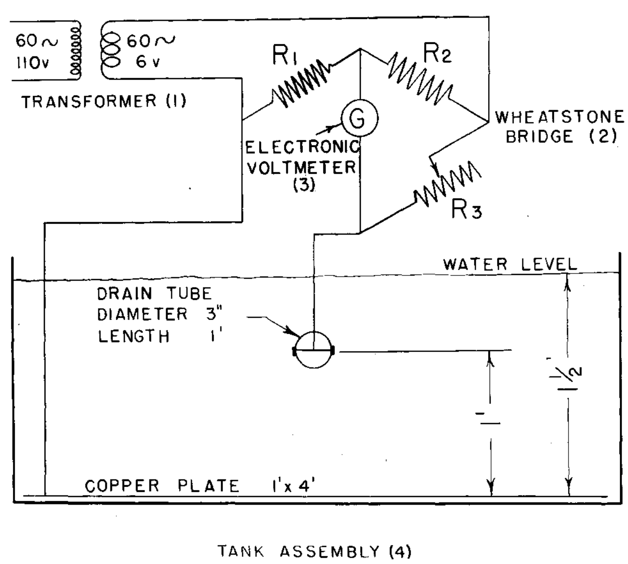

Fig. 18 Circuit Diagram for Electrical Equipment Shown in Fig. 17

In the electric analogue it was not necessary, according to Frevert and Kirkham (12) to measure both voltage and current. One needed only to measure the ratio of these quantities; that is, the resistance between the electrodes representing the drain tube perforations and the copper electrode representing the water table. A Wheatstone bridge was used for making this measurement, an electronic voltmeter being used to balance the circuit. A circuit diagram for the Wheatstone bridge and other equipment is shown in Fig. 18. R_4, the fourth arm in the Wheatstone bridge, was the resistance between the drain tube model and the copper plate. Numbers in parenthesis in Fig. 18 correspond to the numbers in Fig. 17.

Method of procedure

In considering the effect of drain tube perforations on flow it was convenient to use the ratios Q_S/Q_o and Q/Q_o rather than the values of Q_S and Q_o alone. Theoretically, these ratios are (see equations (12), (13), and (14)):

$$\frac{Q_S}{Q_o} = \frac{\log_e (2d/r_w)}{C + \log_e (2d/r_w)} \tag{15}$$

and

$$\frac{Q}{Q_o} = \frac{2}{} \frac{\log_e (2d/r_w)}{C + \log_e (2d/r_w)} \tag{16}$$

It was likewise convenient to use these ratios for experimental tests rather than equations(12), (13), and (14), individually. In the electric case $(d + t - r_w)$ becomes the applied voltage and Q_S the current so that $(d + t - r_w)/Q_S$ corresponds electrically to a resistance which can be denoted R_S, with similar values R and R_o corresponding to Q and Q_o. Thus, if the specific electrical conductivity of the tank water stayed constant throughout all tests, then the expressions

$$\frac{Q_S}{Q_o} = \frac{R_o}{R_S} \tag{17}$$

and

$$\frac{Q}{Q_o} = \frac{R_o}{R} \tag{18}$$

should be identities if the theory was correct. To test equations (17) and (18) the right-hand sides were determined by measuring R_o, R_S, and R (R_o, R_S, and R are R_4 of Fig. 18) and calculating the ratios R_o/R_S and R_o/R. The left-hand sides of equations (17) and (18) were obtained by substituting in the right-hand sides of equations (15) and (16) numerical values of $\underline{d}$ and r_w into the logarithmic terms, and r_w, r_p, $\underline{a}$, and $\underline{m}$ into the constant C as defined in equation (7), and performing the calculations.

When R and R_S were measured, the two lines of plate- and sphere-electrodes were kept in a horizontal plane even though the theory (and tests) showed that the orientation of this plane would be immaterial. In order to minimize changes in specific conductivity of the electrolyte due to temperature changes the tank was kept in a constant temperature room. Tap water was placed in the tank one week before any readings were taken and the water was stirred several times to eliminate temperature variations within the liquid. Alternating current was used in the bridge circuit to prevent polarization. Even so, coatings formed on some of the electrodes and caused a slight increase in resistance. This increase in resistance was more pronounced for the smaller electrodes, and bronze and copper surfaces were affected more than the polished steel balls. To remove any coatings, electrodes of the models were cleaned with steel wool before each reading. It was observed that slight contamination as from grease on fingers could also result in considerable error in measurements.

The data for the electric analogue results were obtained by placing one of the drain tube models shown in Fig. 15 into the tank assembly shown in Fig. 16. In using drain tube models 2 and 4 spacing of electrodes was obtained by connecting the proper combination of leads to the Wheatstone bridge. Each pair of opposite electrodes, one on each side, was connected by a lead wire so that spacings of 1, 2, 4, 6, and 12 inches could be obtained. For example, a spacing of 12 inches was obtained by using a pair of electrodes near the center of the model and shifting the model along its axis in the tank assembly so as to place the electrodes 6 inches from the side walls of the tank assembly. With the proper leads connected the resistance of the electrodes was determined with the electronic voltmeter and the Wheatstone bridge. When a minimum reading was obtained on the electronic voltmeter a balance on the bridge was obtained and the resistance was determined. Normally, a reading of less than 1/100 volt on the voltmeter was obtained in securing the proper balance. The resistance value and the size of electrodes affected the minimum voltmeter reading.

A Bouyoucos bridge was first used to determine resistance, but it did not give consistent results and the values were erroneous. A second Bouyoucos bridge was tried and it was found that the readings would not check with the first bridge. A 110-volt 1000-cycle source of current was also tried, but readings could not be taken as accurately as with the 6-volt 60-cycle current.

Results

Measured and theoretical values for sphere- and plate-electrodes are presented in Fig. 19. The experimental points are the average values of three observations as shown in Table 2. The curved lines were drawn from data which are presented in Table 3. Solid lines are theoretical curves obtained on the assumption that drain tube perforations are equivalent to spherical openings in the soil. Dashed lines are theoretical curves based on the assumption that the perforations are flat, circular openings in drain tubes. Although the theoretical curves are somewhat higher, generally, than the experimental points, it is felt that the results serve to verify the theory and that therefore the theoretical equations can be used with confidence to calculate rather extensive results for practical use. These results, representing 672 independent calculations of the right-hand member of equation (16), are brought together in Table 3 and Figs. 20 and 21. The calculations were made for diameters of tubes of 2, 4, 6, and 12 inches, for 2, 4, and 8 rows of holes, for 1/4- and 1/2-inch diameter perforations, and for depths of 1, 2, 4, and 8 feet in all possible combinations. The theoretical points for each combination were calculated for spacings of 0, 1, 2, 3, 6, 12, and 24 inches. For plates the theoretical curves, as shown in Fig. 19, are low at zero spacing of perforations compared to experimental values. In noting these low values one should remember that the theoretical curves for plates are not valid at spacings less than 2 to 3 inches. Although the theory begins to break down at narrow spacings, equation (16) has been used as an approximation to obtain the curves shown in Figs. 20 and 21 for all spacings. In Table 3 for r_w = 1 and 2 several values of Q_S/Q_O were greater than unity when the number of holes per foot was infinite (longitudinal slots in the tube). In practice it is doubtful if such close spacings will be used.

Table 2

Experimental Values of Q_S/Q_o and Q/Q_o for Spheres and Plates

(r_w = 3", m = 2, and d = 2')

		Spheres			Plates		
r_p	Hole spacing	R_o	R_S	Q_S/Q_o	R_o	R	Q/Q_o
inches	inches	ohms	ohms		ohms	ohms	
1/4	0	26.7	38.0	0.703	24.5	44.5	0.551
		26.5	38.0	0.697	25.6	46.6	0.549
		24.9	35.4	0.703	24.9	45.5	0.547
			Avg.	0.701		Avg.	0.549
1/4	2	26.7	46.0	0.580	24.5	66.7	0.367
		26.5	47.0	0.564	25.6	69.7	0.367
		24.9	44.5	0.560	24.9	68.5	0.364
			Avg.	0.568		Avg.	0.366
1/4	4	26.7	63.0	0.424	24.7	104.0	0.238
		26.5	63.0	0.421	24.4	106.0	0.230
		24.9	59.5	0.418	24.7	105.0	0.235
			Avg.	0.421		Avg.	0.234
1/4	8	26.7	105.0	0.254	24.5	186.0	0.132
		26.5	105.0	0.252	25.6	195.0	0.131
		24.9	92.5	0.269	24.9	192.0	0.130
			Avg.	0.258		Avg.	0.131
1/4	12	26.7	142.0	0.188	24.5	260.0	0.094
		26.5	140.0	0.189	25.6	276.0	0.093
		24.9	130.0	0.192	24.9	272.0	0.092
			Avg.	0.190		Avg.	0.093
1/4	24	26.7	260.0	0.103	24.5	490.0	0.050
		26.5	250.0	0.106	25.6	533.0	0.048
		24.9	235.0	0.106	24.9	530.0	0.047
			Avg.	0.105		Avg.	0.048

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Table 2 (Cont'd)

r_p	Hole spacing	Spheres R_o	Spheres R_S	Spheres Q_S/Q_o	Plates R_o	Plates R	Plates Q/Q_o
inches	inches	ohms	ohms		ohms	ohms	
1/8	0	26.7	41.0	0.651	25.6	51.5	0.497
		26.5	41.0	0.646	26.5	53.0	0.500
		24.9	38.6	0.645	24.9	50.3	0.495
			Avg.	0.647		Avg.	0.497
1/8	2	26.7	67.0	0.399	25.6	117.0	0.219
		26.5	65.0	0.408	26.5	122.0	0.217
		24.9	62.0	0.402	24.9	116.0	0.215
			Avg.	0.403		Avg.	0.217
1/8	4	26.7	102.0	0.262	26.7	220.0	0.121
		26.5	99.0	0.268	26.5	226.0	0.117
		24.9	94.0	0.265	24.9	199.0	0.125
			Avg.	0.265		Avg.	0.121
1/8	8	26.7	173.0	0.154	26.7	397.0	0.067
		26.5	175.0	0.151	26.5	400.0	0.066
		24.9	168.0	0.148	24.9	370.0	0.067
			Avg.	0.151		Avg.	0.067
1/8	12	26.7	250.0	0.107	26.7	580.0	0.046
		26.5	250.0	0.106	26.5	590.0	0.045
		24.9	237.0	0.105	24.9	545.0	0.046
			Avg.	0.106		Avg.	0.046
1/8	24	26.7	470.0	0.057	26.7	1110.0	0.024
		26.5	458.0	0.058	26.5	1110.0	0.024
		24.9	440.0	0.057	24.9	1050.0	0.024
			Avg.	0.057		Avg.	0.024

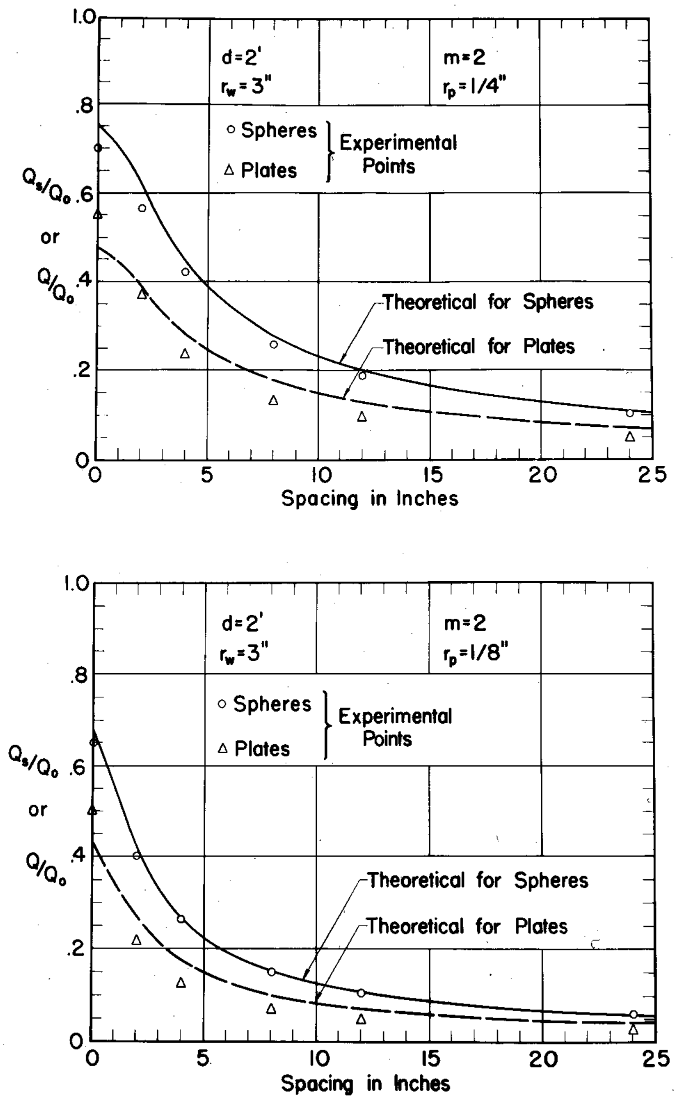

Fig. 19 Comparison of Experimental and Theoretical Values of the Effect of Perforations in Drain Tubes

Table 3

Average Theoretical Values of Q_S/Q_o, Q/Q_o, and Q and Theoretical Values of Q_S/Q_o for 2, 4, and 8 Rows of Perforations

r_w	r_p	Depth	Holes per foot	Q_S/Q_o m = 2	Q_S/Q_o m = 4	Q_S/Q_o m = 8	Q_S/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.126	0.126	-----	0.126	0.080	0.159
			4	0.231	0.230	0.229	0.230	0.146	0.289
			8	0.387	0.384	0.376	0.382	0.243	0.481
1	1/8	1	16	0.570	0.576	0.552	0.566	0.360	0.712
			24	0.666	0.687	0.660	0.671	0.427	0.845
			48	-----	0.836	0.828	0.832	0.530	1.047
			Infinite	0.821	0.948	1.000			
			2	0.149	0.149	-----	0.149	0.095	0.308
			4	0.267	0.266	0.265	0.266	0.169	0.550
			8	0.435	0.432	0.423	0.430	0.274	0.889
1	1/8	2	16	0.622	0.623	0.600	0.615	0.392	1.271
			24	0.709	0.728	0.705	0.714	0.455	1.475
			48	-----	0.862	0.854	0.858	0.546	1.773
			Infinite	0.848	0.957	1.000			
			2	0.172	0.171	-----	0.172	0.109	0.603
			4	0.301	0.300	0.298	0.300	0.191	1.052
			8	0.475	0.473	0.464	0.471	0.300	1.651
1	1/8	4	16	0.660	0.661	0.639	0.653	0.416	2.289
			24	0.742	0.759	0.737	0.746	0.475	2.615
			48	-----	0.880	0.873	0.877	0.558	3.074
			Infinite	0.868	0.963	1.000			

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Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_s/Q_o m = 2	Q_s/Q_o m = 4	Q_s/Q_o m = 8	Q_s/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.193	0.192	-----	0.193	0.123	1.175
			4	0.331	0.330	0.329	0.330	0.210	2.009
			8	0.511	0.508	0.499	0.506	0.322	3.080
1	1/8	8	16	0.690	0.692	0.671	0.684	0.435	4.163
			24	0.768	0.784	0.764	0.772	0.491	4.699
			48	-----	0.894	0.888	0.891	0.567	5.423
			Infinite	0.884	0.968	1.000			
			2	0.240	0.240	-----	0.240	0.153	0.302
			4	0.408	0.406	0.402	0.405	0.258	0.510
			8	0.610	0.603	0.583	0.599	0.381	0.754
1	1/4	1	16	0.792	0.790	0.747	0.776	0.494	0.977
			24	0.846	0.876	0.835	0.852	0.542	1.072
			48	-----	0.965	0.952	0.959	0.611	1.207
			Infinite	0.902	1.000	1.028			
			2	0.278	0.278	-----	0.278	0.177	0.574
			4	0.457	0.454	0.450	0.454	0.289	0.938
			8	0.655	0.649	0.630	0.645	0.411	1.333
1	1/4	2	16	0.825	0.821	0.783	0.810	0.516	1.674
			24	0.870	0.896	0.862	0.876	0.558	1.810
			48	-----	0.971	0.960	0.966	0.615	1.996
			Infinite	0.918	1.000	1.023			

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Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_S/Q_o m = 2	Q_S/Q_o m = 4	Q_S/Q_o m = 8	Q_S/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.313	0.312	-----	0.313	0.199	1.097
			4	0.498	0.495	0.491	0.495	0.315	1.735
			8	0.692	0.685	0.668	0.682	0.434	2.391
1	1/4	4	16	0.847	0.844	0.809	0.833	0.530	2.920
			24	0.888	0.911	0.878	0.892	0.568	3.127
			48	-----	0.975	0.966	0.971	0.618	3.404
			Infinite	0.929	1.000	1.019			
			2	0.344	0.343	-----	0.344	0.219	2.094
			4	0.533	0.530	0.527	0.530	0.337	3.226
			8	0.721	0.715	0.698	0.711	0.453	4.328
1	1/4	8	16	0.861	0.862	0.830	0.851	0.542	5.180
			24	0.901	0.921	0.892	0.905	0.576	5.508
			48	-----	0.979	0.970	0.975	0.621	5.934
			Infinite	0.938	1.000	1.017			
			2	0.099	0.099	-----	0.099	0.063	0.159
			4	0.184	0.183	0.181	0.183	0.117	0.295
			8	0.315	0.317	0.308	0.313	0.199	0.504
2	1/8	1	16	0.480	0.496	0.484	0.487	0.310	0.784
			24	0.562	0.605	0.600	0.589	0.375	0.948
			48	-----	0.758	0.774	0.766	0.488	1.233
			Infinite	0.705	0.878	0.966			

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Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_S/Q_o m = 2	Q_S/Q_o m = 4	Q_S/Q_o m = 8	Q_S/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.123	0.123	-----	0.123	0.078	0.310
			4	0.224	0.223	0.220	0.222	0.141	0.559
			8	0.371	0.373	0.363	0.369	0.235	0.929
2	1/8	2	16	0.540	0.557	0.546	0.548	0.349	1.379
			24	0.621	0.662	0.655	0.646	0.411	1.626
			48	-----	0.800	0.814	0.807	0.514	2.031
			Infinite	0.753	0.902	0.973			
			2	0.146	0.146	-----	0.146	0.093	0.603
			4	0.260	0.259	0.256	0.258	0.164	1.066
			8	0.418	0.420	0.410	0.416	0.265	1.719
2	1/8	4	16	0.594	0.605	0.594	0.598	0.381	2.472
			24	0.667	0.705	0.699	0.690	0.439	2.852
			48	-----	0.830	0.842	0.836	0.532	3.455
			Infinite	0.788	0.918	0.978			
			2	0.168	0.168	-----	0.168	0.107	1.178
			4	0.293	0.291	0.288	0.291	0.185	2.040
			8	0.458	0.461	0.450	0.456	0.290	3.197
2	1/8	8	16	0.630	0.644	0.633	0.636	0.405	4.459
			24	0.702	0.738	0.735	0.725	0.462	5.083
			48	-----	0.852	0.863	0.858	0.546	6.015
			Infinite	0.814	0.929	0.981			

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Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_s/Q_o m = 2	Q_s/Q_o m = 4	Q_s/Q_o m = 8	Q_s/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.190	0.189	-----	0.190	0.121	0.306
			4	0.330	0.328	0.320	0.326	0.208	0.525
			8	0.510	0.515	0.491	0.505	0.321	0.813
2	1/4	1	16	0.680	0.709	0.685	0.691	0.440	1.112
			24	0.729	0.800	0.790	0.773	0.492	1.244
			48	-----	0.896	0.917	0.907	0.577	1.460
			Infinite	0.782	0.935	1.000			
			2	0.231	0.230	-----	0.231	0.147	0.581
			4	0.387	0.384	0.376	0.382	0.243	0.962
			8	0.571	0.576	0.552	0.566	0.360	1.425
2	1/4	2	16	0.727	0.757	0.735	0.740	0.471	1.863
			24	0.775	0.836	0.827	0.813	0.518	2.046
			48	-----	0.917	0.934	0.926	0.590	2.331
			Infinite	0.821	0.948	1.000			
			2	0.267	0.266	-----	0.267	0.170	1.104
			4	0.435	0.432	0.423	0.430	0.274	1.777
			8	0.618	0.623	0.600	0.614	0.391	2.538
2	1/4	4	16	0.767	0.791	0.772	0.777	0.495	3.211
			24	0.807	0.862	0.855	0.841	0.535	3.476
			48	-----	0.931	0.945	0.938	0.597	3.877
			Infinite	0.848	0.957	1.000			

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Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_s/Q_o m = 2	Q_s/Q_o m = 4	Q_s/Q_o m = 8	Q_s/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.301	0.300	-----	0.301	0.192	2.110
			4	0.475	0.473	0.464	0.471	0.300	3.302
			8	0.656	0.661	0.639	0.652	0.415	4.571
2	1/4	8	16	0.790	0.817	0.800	0.802	0.511	5.623
			24	0.832	0.880	0.874	0.862	0.549	6.043
			48	-----	0.941	0.953	0.947	0.603	6.639
			Infinite	0.868	0.963	1.000			
			2	0.083	0.082	-----	0.083	0.053	0.160
			4	0.154	0.156	0.153	0.154	0.098	0.296
			8	0.269	0.274	0.269	0.271	0.173	0.521
3	1/8	1	16	0.413	0.442	0.433	0.429	0.273	0.825
			24	0.483	0.546	0.548	0.526	0.335	1.012
			48	-----	0.681	0.727	0.704	0.448	1.354
			Infinite	0.619	0.811	0.932			
			2	0.108	0.108	-----	0.108	0.069	0.312
			4	0.196	0.198	0.195	0.196	0.125	0.566
			8	0.329	0.334	0.329	0.331	0.211	0.955
3	1/8	2	16	0.485	0.514	0.504	0.501	0.319	1.446
			24	0.555	0.616	0.619	0.597	0.380	1.723
			48	-----	0.740	0.781	0.761	0.484	2.196
			Infinite	0.684	0.851	0.948			

(Continued on next page)

Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_S/Q_o m = 2	Q_S/Q_o m = 4	Q_S/Q_o m = 8	Q_S/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.131	0.132	-----	0.132	0.084	0.609
			4	0.233	0.235	0.233	0.234	0.149	1.080
			8	0.380	0.386	0.380	0.382	0.243	1.764
3	1/8	4	16	0.538	0.568	0.560	0.555	0.353	2.562
			24	0.608	0.667	0.668	0.648	0.413	2.992
			48	-----	0.780	0.816	0.798	0.508	3.684
			Infinite	0.729	0.877	0.958			
			2	0.154	0.155	-----	0.155	0.099	1.193
			4	0.267	0.270	0.265	0.267	0.170	2.054
			8	0.423	0.430	0.423	0.425	0.271	3.270
3	1/8	8	16	0.585	0.612	0.603	0.600	0.382	4.617
			24	0.652	0.706	0.708	0.689	0.439	5.301
			48	-----	0.810	0.842	0.826	0.526	6.355
			Infinite	0.763	0.895	0.965			
			2	0.158	0.162	-----	0.160	0.102	0.308
			4	0.284	0.284	0.274	0.281	0.179	0.541
			8	0.453	0.461	0.442	0.452	0.288	0.869
3	1/4	1	16	0.595	0.649	0.638	0.627	0.399	1.206
			24	0.638	0.740	0.750	0.709	0.451	1.364
			48	-----	0.834	0.879	0.857	0.546	1.648
			Infinite	0.697	0.886	0.970			

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Table 3 (Cont'd)

r_w inches	r_p inches	Depth feet	Holes per foot	Q_s/Q_o m = 2	Q_s/Q_o m = 4	Q_s/Q_o m = 8	Q_s/Q_o Avg.	Q/Q_o Avg.	Q Avg. cu.ft./ft./day
			2	0.200	0.202	-----	0.201	0.128	0.580
			4	0.345	0.345	0.335	0.342	0.218	0.987
			8	0.524	0.533	0.514	0.524	0.334	1.512
3	1/4	2	16	0.660	0.711	0.702	0.691	0.440	1.994
			24	0.702	0.791	0.800	0.764	0.486	2.204
			48	-----	0.870	0.907	0.889	0.566	2.565
			Infinite	0.754	0.912	0.978			
			2	0.239	0.240	-----	0.240	0.153	1.108
			4	0.398	0.398	0.386	0.394	0.251	1.819
			8	0.579	0.587	0.568	0.578	0.368	2.668
3	1/4	4	16	0.710	0.754	0.746	0.737	0.469	3.403
			24	0.746	0.826	0.828	0.800	0.509	3.693
			48	-----	0.893	0.924	0.909	0.579	4.197
			Infinite	0.793	0.928	0.982			
			2	0.273	0.275	-----	0.274	0.174	2.108
			4	0.442	0.442	0.429	0.438	0.279	3.370
			8	0.623	0.631	0.613	0.622	0.396	4.786
3	1/4	8	16	0.746	0.787	0.779	0.771	0.491	5.932
			24	0.779	0.850	0.855	0.828	0.527	6.371
			48	-----	0.910	0.935	0.923	0.588	7.102
			Infinite	0.822	0.938	0.985			

(Continued on next page)

Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_S/Q_o m = 2	Q_S/Q_o m = 4	Q_S/Q_o m = 8	Q_S/Q_o Avg.	Q/Q_o Avg.	Q Avg.
inches	inches	feet							cu.ft./ft./day
			2	0.056	0.057	-----	0.057	0.036	0.164
			4	0.106	0.108	0.108	0.107	0.068	0.309
			8	0.188	0.196	0.195	0.193	0.123	0.557
6	1/8	1	16	0.295	0.331	0.335	0.320	0.204	0.923
			24	0.354	0.421	0.436	0.404	0.257	1.166
			48	-----	0.553	0.619	0.586	0.373	1.691
			Infinite	0.459	0.686	0.862			
			2	0.082	0.082	-----	0.082	0.052	0.315
			4	0.151	0.153	0.152	0.152	0.097	0.585
			8	0.257	0.268	0.266	0.264	0.168	1.016
6	1/8	2	16	0.387	0.425	0.430	0.414	0.264	1.593
			24	0.452	0.521	0.537	0.503	0.320	1.935
			48	-----	0.650	0.709	0.680	0.433	2.616
			Infinite	0.560	0.767	0.903			
			2	0.106	0.108	-----	0.107	0.068	0.617
			4	0.191	0.195	0.194	0.193	0.123	1.114
			8	0.316	0.328	0.326	0.323	0.206	1.864
6	1/8	4	16	0.462	0.497	0.502	0.487	0.310	2.810
			24	0.523	0.592	0.606	0.574	0.365	3.312
			48	-----	0.713	0.765	0.739	0.470	4.265
			Infinite	0.630	0.814	0.926			

(Continued on next page)

Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_s/Q_o				Q/Q_o	Q
				m = 2	m = 4	m = 8	Avg.	Avg.	Avg.
inches	inches	feet							cu.ft./ ft./day
			2	0.129	0.131	-----	0.130	0.083	1.200
			4	0.228	0.232	0.230	0.230	0.146	2.124
			8	0.366	0.378	0.377	0.374	0.238	3.453
6	1/8	8	16	0.515	0.552	0.557	0.541	0.344	4.995
			24	0.578	0.644	0.658	0.627	0.399	5.789
			48	-----	0.755	0.802	0.779	0.496	7.193
			Infinite	0.680	0.845	0.938			
			2	0.109	0.111	-----	0.110	0.070	0.317
			4	0.197	0.203	0.200	0.200	0.127	0.577
			8	0.321	0.342	0.341	0.335	0.213	0.967
6	1/4	1	16	0.441	0.513	0.526	0.493	0.314	1.423
			24	0.476	0.610	0.635	0.574	0.365	1.656
			48	-----	0.695	0.786	0.741	0.472	2.138
			Infinite	0.522	0.751	0.913			
			2	0.154	0.156	-----	0.155	0.099	0.596
			4	0.269	0.276	0.271	0.272	0.173	1.046
			8	0.415	0.438	0.436	0.430	0.274	1.654
6	1/4	2	16	0.542	0.612	0.625	0.593	0.378	2.281
			24	0.577	0.701	0.723	0.667	0.425	2.566
			48	-----	0.774	0.846	0.810	0.516	3.116
			Infinite	0.621	0.819	0.940			

(Continued on next page)

Table 3 (Cont'd)

r_w	r_p	Depth	Holes per foot	Q_s/Q_o				Q/Q_o	Q
				m = 2	m = 4	m = 8	Avg.	Avg.	Avg.
inches	inches	feet							cu.ft./ ft./day
			2	0.196	0.197	-----	0.197	0.125	1.137
			4	0.329	0.338	0.331	0.333	0.212	1.922
			8	0.485	0.509	0.508	0.501	0.319	2.891
6	1/4	4	16	0.612	0.677	0.690	0.660	0.420	3.809
			24	0.645	0.758	0.777	0.727	0.463	4.195
			48	-----	0.820	0.880	0.850	0.541	4.905
			Infinite	0.686	0.858	0.955			
			2	0.233	0.234	-----	0.234	0.149	2.161
			4	0.381	0.389	0.381	0.384	0.244	3.546
			8	0.542	0.565	0.563	0.557	0.355	5.143
6	1/4	8	16	0.662	0.724	0.736	0.707	0.450	6.528
			24	0.694	0.796	0.812	0.767	0.488	7.082
			48	-----	0.852	0.902	0.877	0.558	8.098
			Infinite	0.732	0.884	0.963			

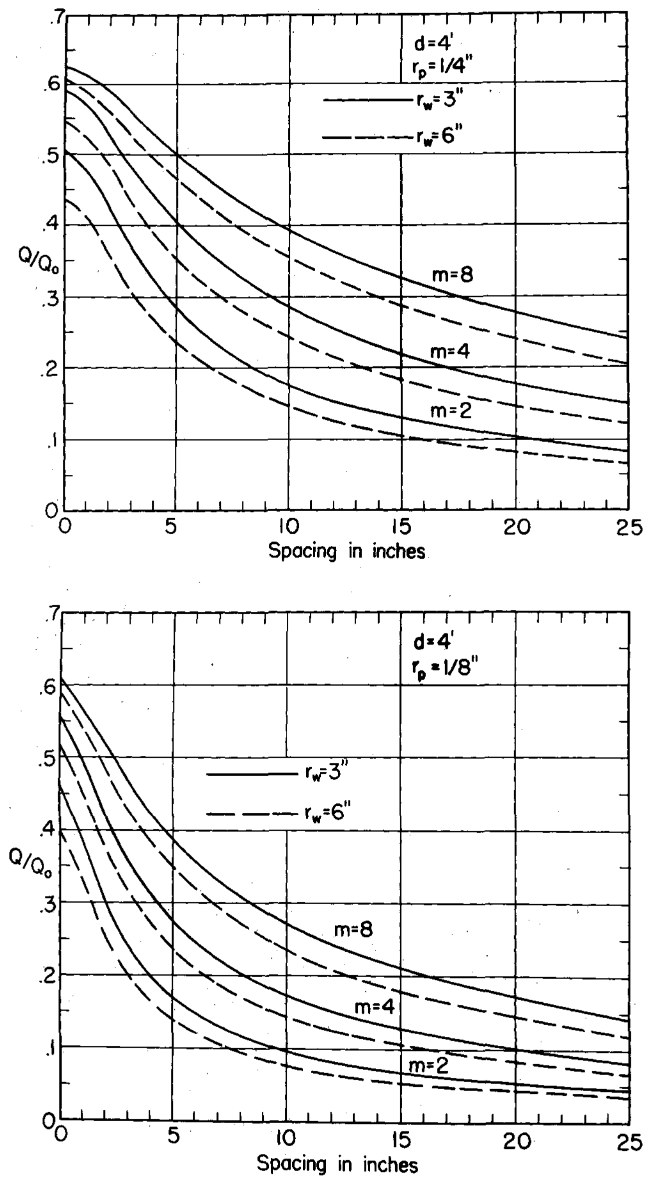

Fig. 20 Theoretical Values of Q/Q_o Versus Spacing of Perforations

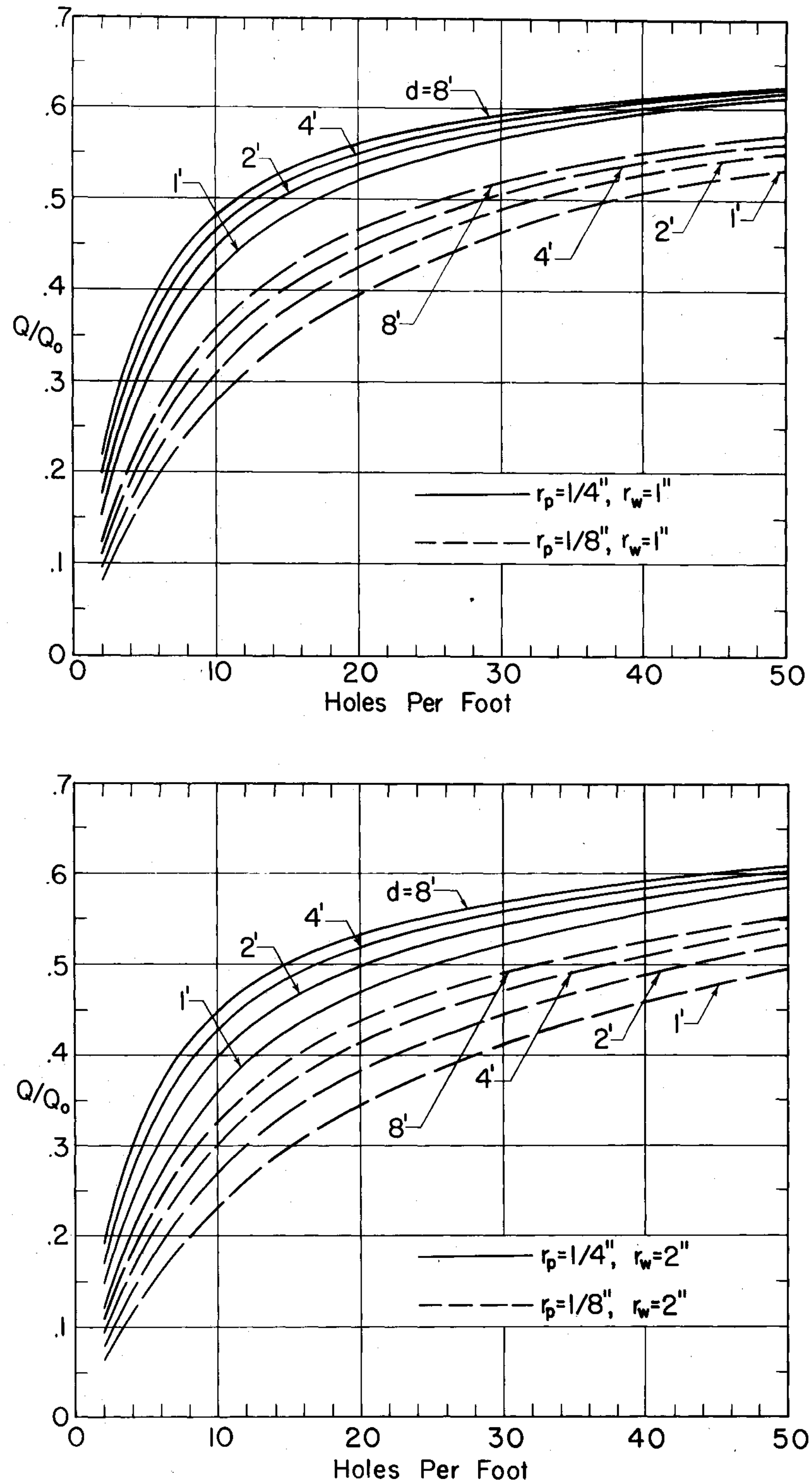

Fig. 21a Theoretical Values of Q/Q_o Versus Number of Perforations per Foot of Drain Tube for Tubes 2 and 4 Inches in Diameter

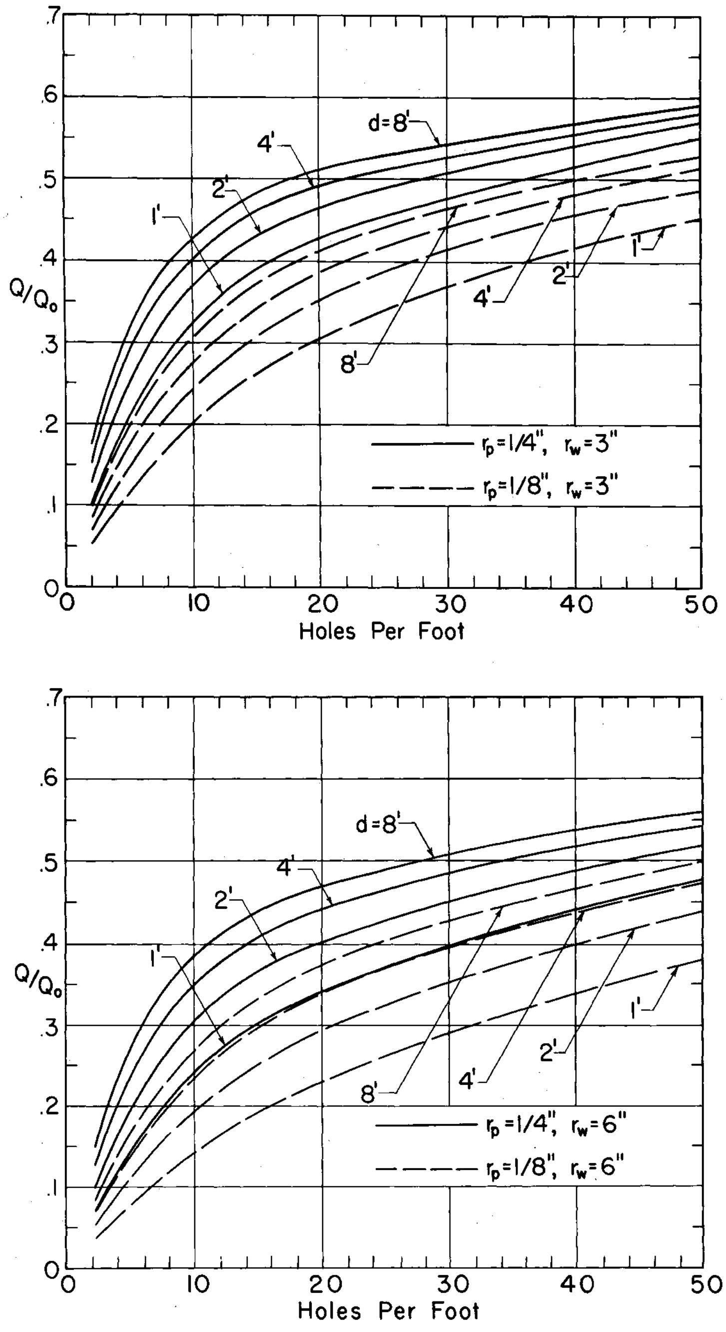

Fig. 21b Theoretical Values of Q/Q_o Versus Number of Perforations per Foot of Drain Tube for Tubes 6 and 12 Inches in Diameter

Fig. 20 presents the data in terms of spacing and Fig. 21 in terms of holes per foot. The compact presentation of data in Fig. 21, which comprises all results, stems from the observation in Table 3 and Fig. 20 that equal numbers of holes per foot yield, very nearly, the same total flow per foot regardless of the number of rows of holes.* The data for the curves in Fig. 21, except for values of Q/Q_o for 2 and 48 holes per foot, are average values of Q/Q_o for $m = 2$, 4, and 8. For 2 holes per foot Q/Q_o is the average for $m = 2$ and 4, and for 48 holes per foot the average for $m = 4$ and 8. (See Table 3.)

Curves for the absolute values of Q are given in Fig. 22 and the data in Table 3. The curves were obtained from those of Fig. 21 by multiplication of the ordinates by $Q_o = 2\pi K(d + t - r_w)/\log_e (2d/r_w)$. In the calculations $\underline{K}$ is taken as 1 foot per day, $\underline{t}$ is taken equal to zero, and r_w is omitted from the expression $(d + t - r_w)$. The term r_w is omitted since it is assumed that here the drains are running about half full, rather than full.

It is of interest to compare flow into a drain tube having circular perforations, with flow into more

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*This observation is not quite true for large diameter drains. For example, for the extreme case of a 12-inch diameter drain at 1-foot depth with 24 1/2-inch holes per foot, the value of Q/Q_o for 2 rows of holes is 0.303, for 4 rows of holes 0.388, and for 8 rows of holes 0.404. The average value here is 0.365 and the deviation from the average value is, therefore, approximately (0.365 - 0.303) 100 = 6.2 percent.

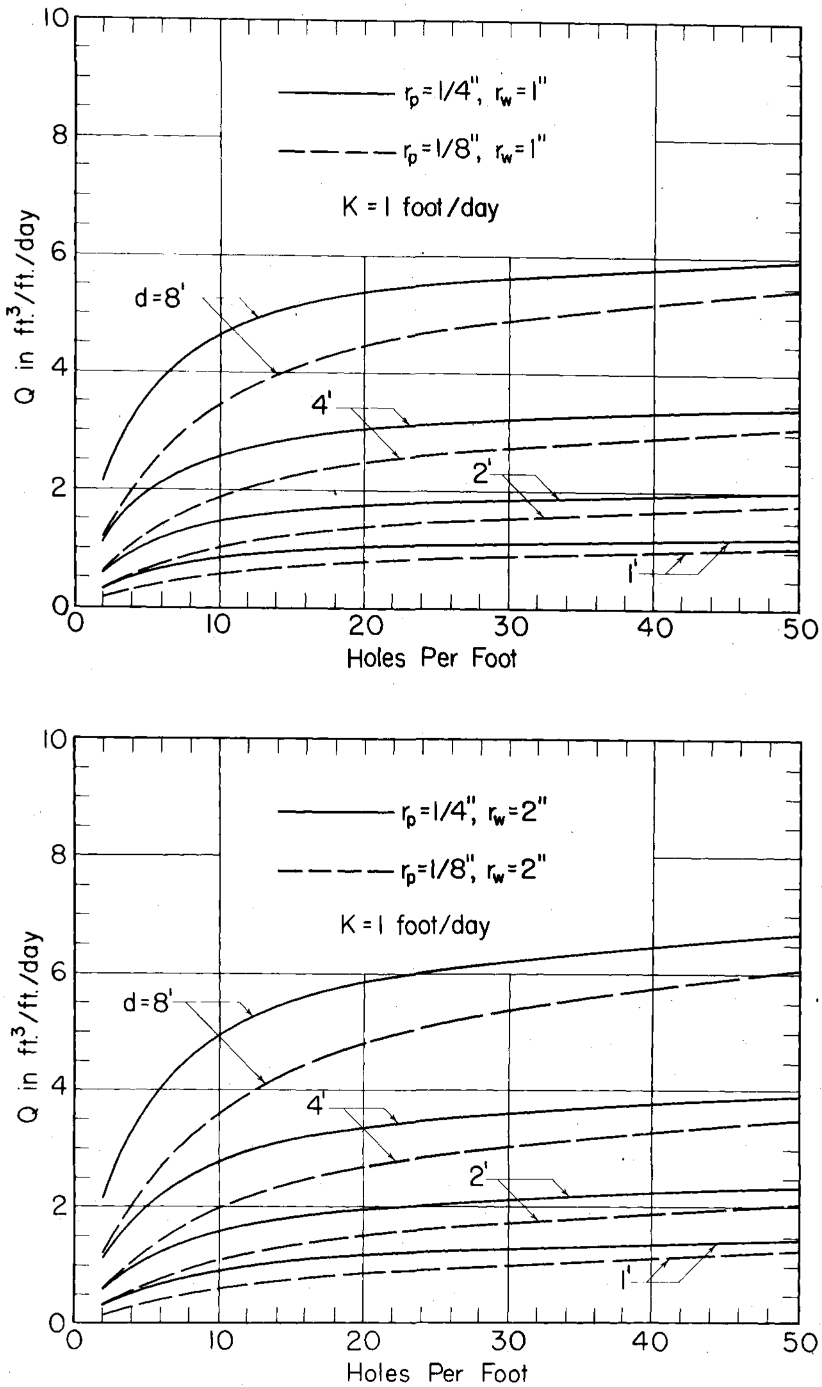
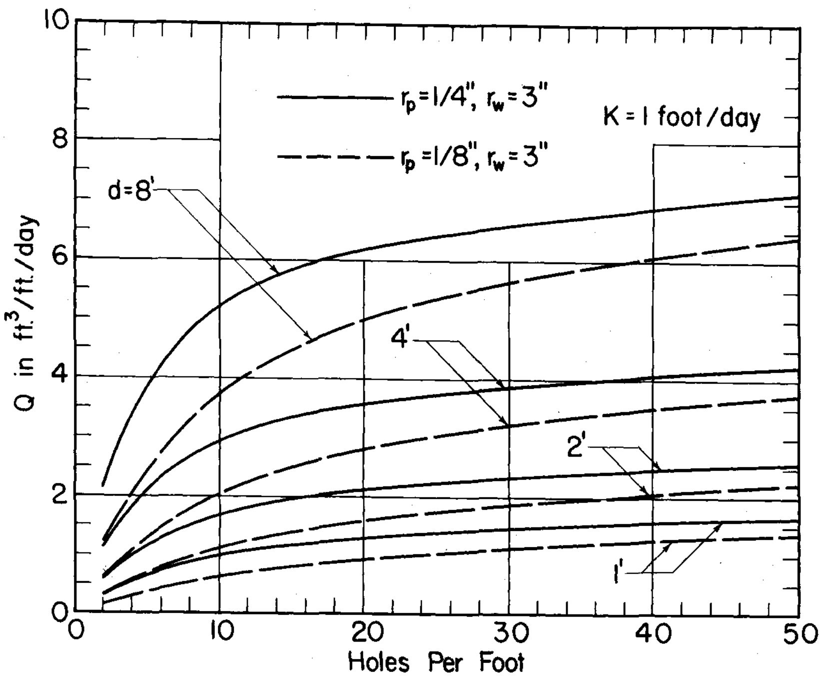


Fig. 22a Flow Q into Perforated Drain Tubes 2 and 4 Inches in Diameter in Cubic Feet per Day per Foot of Length



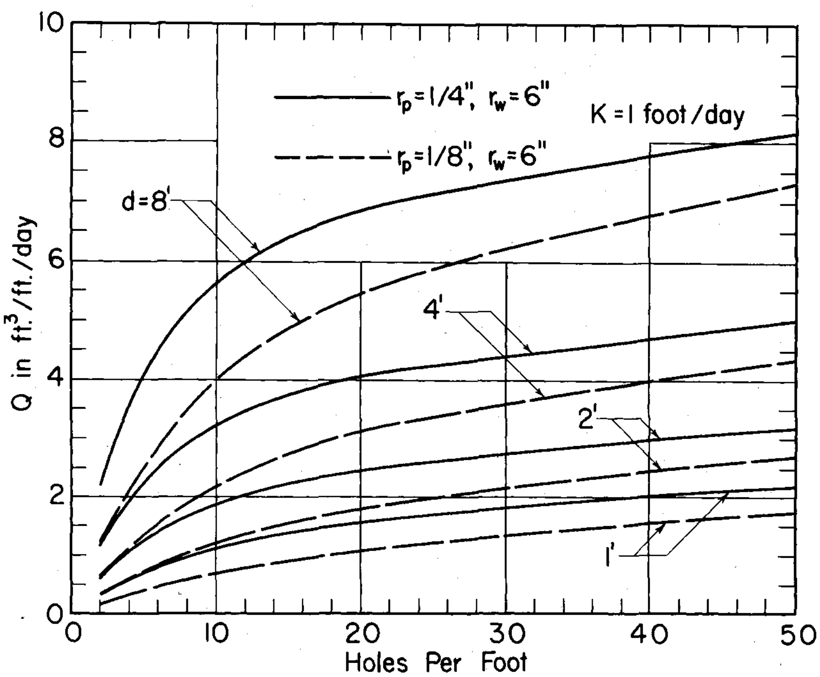


Fig. 22b Flow Q into Perforated Drain Tubes 6 and 12 Inches in Diameter in Cubic Feet per Day per Foot of Length

conventional drains having cracks between the unit lengths of impermeable drain pipe. Kirkham (21) has shown in the latter case that if one takes the flow into the completely pervious drain tube as 1.000, then the flow into a 4-foot deep, 6-inch outside diameter drain tube consisting of a series of 1-foot long impermeable lengths is 0.433, 0.485, and 0.555 for crack widths between the 1-foot lengths of 1/8, 1/4, and 1/2 inch, respectively. For the same conditions, except for a depth of 2 feet, the values are 0.379, 0.430, and 0.491. Using the electric analogue method the latter three values were in agreement to less than 3 per cent.

The effect of having a single row of perforations on the top or on the bottom of the drain was determined by the electric analogue and the results are shown in Table 4 and Fig. 23. The solid curve in Fig. 23 shows experimental data from Table 2 for two rows of perforations. The dashed curves represent the experimental data for one row of holes on top or on the bottom of the drain tube. The flow as indicated by the ratio Q/Q_o was slightly greater when the perforations were on top than when on the bottom. The difference was greater as the spacing between perforations was decreased or as the number of holes per foot was increased. At a spacing of 24 inches the difference was not measurable. Although the data for $r_p = 1/8$ inch are not shown graphically, the results given in Table 4 are similar to those in Fig. 23. The difference between holes on the top and on the bottom of

Table 4

Experimental Values of Q/Q_o for Holes on Top or on Bottom of Drain Tube

(r_w = 3", m = 1, and d = 2')

r_p	Hole spacing	Holes on top			Holes on bottom		
		R_o	R	Q/Q_o	R_o	R	Q/Q_o
inches	inches	ohms	ohms		ohms	ohms	
1/4	0	18.0	55.0	0.327	18.0	63.0	0.286
		19.0	58.0	0.328	19.0	65.0	0.292
			Avg.	0.327		Avg.	0.289
1/4	2	18.0	89.0	0.202	18.0	96.0	0.188
		19.0	92.0	0.207	19.0	99.0	0.192
			Avg.	0.204		Avg.	0.190
1/4	4	18.0	130.0	0.138	18.0	137.0	0.131
		19.0	147.0	0.129	19.0	156.0	0.122
			Avg.	0.134		Avg.	0.127
1/4	8	18.0	259.0	0.069	18.0	269.0	0.067
		19.0	270.0	0.070	19.0	280.0	0.068
			Avg.	0.070		Avg.	0.067
1/4	12	18.0	359.0	0.050	18.0	368.0	0.049
		19.0	360.0	0.053	19.0	367.0	0.052
			Avg.	0.051		Avg.	0.050
1/4	24	18.0	730.0	0.025	18.0	740.0	0.024
		19.0	750.0	0.025	19.0	760.0	0.025
			Avg.	0.025		Avg.	0.025

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Table 4 (Cont'd)

r_p	Hole spacing	Holes on top R_o	Holes on top R	Holes on top Q/Q_o	Holes on bottom R_o	Holes on bottom R	Holes on bottom Q/Q_o
inches	inches	ohms	ohms		ohms	ohms	
1/8	0	18.0	65.0	0.277	18.0	73.0	0.247
		19.0	69.0	0.275	19.0	76.0	0.250
			Avg.	0.276		Avg.	0.248
1/8	2	18.0	158.0	0.114	18.0	170.0	0.106
		19.0	165.0	0.115	19.0	173.0	0.110
			Avg.	0.114		Avg.	0.108
1/8	4	18.0	283.0	0.064	18.0	293.0	0.061
		19.0	295.0	0.064	19.0	312.0	0.061
			Avg.	0.064		Avg.	0.061
1/8	8	18.0	520.0	0.035	18.0	533.0	0.034
		19.0	530.0	0.036	19.0	562.0	0.034
			Avg.	0.035		Avg.	0.034
1/8	12	18.0	770.0	0.023	18.0	782.0	0.023
		19.0	810.0	0.023	19.0	830.0	0.023
			Avg.	0.023		Avg.	0.023
1/8	24	18.0	1430.0	0.013	18.0	1460.0	0.012
		19.0	1510.0	0.013	19.0	1560.0	0.012
			Avg.	0.013		Avg.	0.012

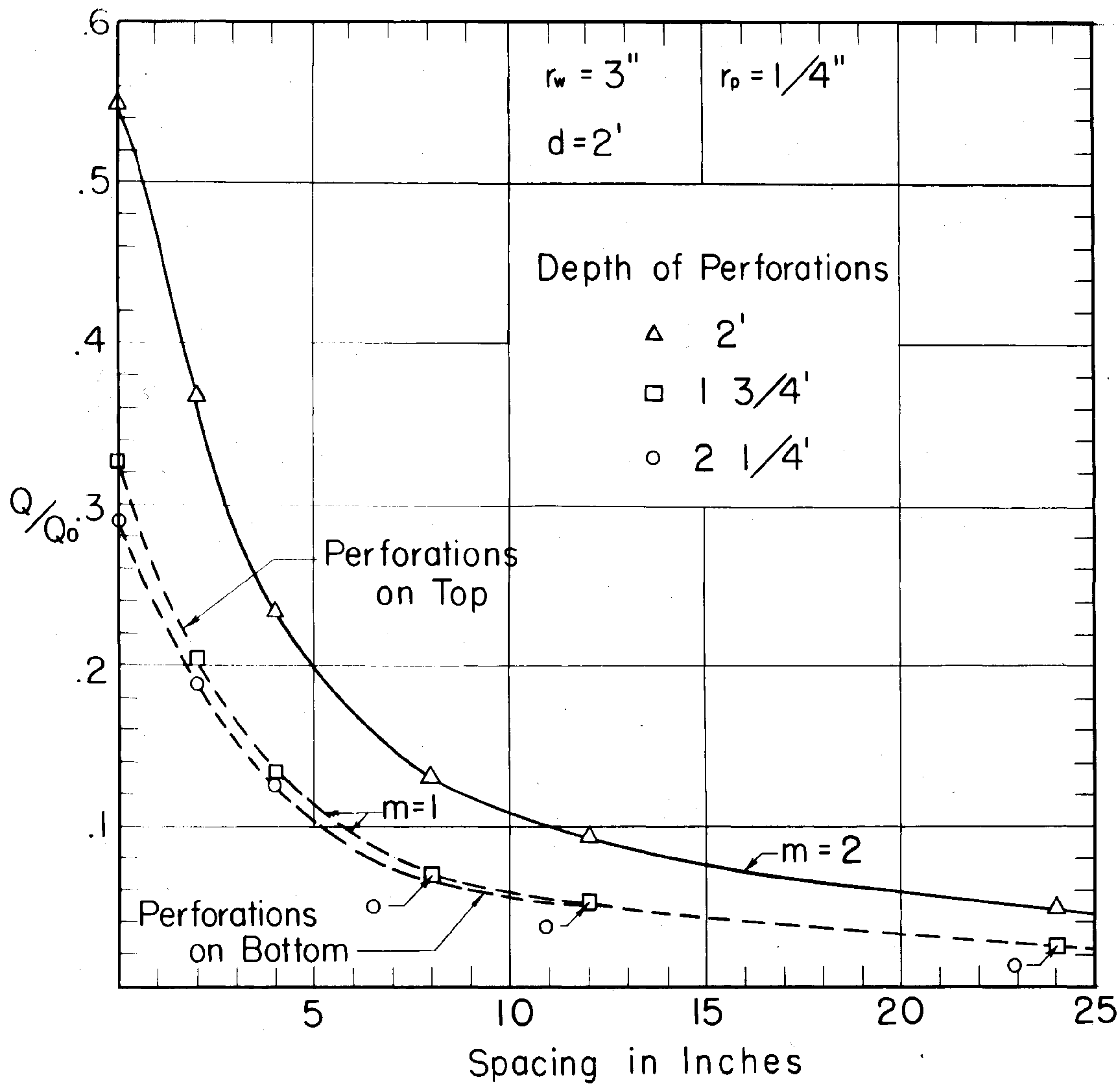


Fig. 23 Experimental Values of Q/Q_o for Holes on Top or on Bottom of Drain Tube

the drain tube was not as great for small perforations as for larger perforations.

The effect of perforations on flow into subsurface drain tubes was determined only for a completely saturated soil of uniform permeability. Since the theoretical equations were in excellent agreement with the values obtained with the electric analogue, the data were verified for a limited number of conditions. As these conclusions are based on analytical procedures and the electric analogue, observations on field installations should be made to provide further check on the theory. Since saturated conditions seldom occur in the field, the effect of perforations will generally be less than indicated.

Effect of Deviations from True Grade on the Performance of Small Perforated Drain Tubes

Subsurface drainage with small drain tubes on slopes of 1 per cent or greater present installation problems not generally encountered with larger diameter drains. It does not appear practical to install small-size drains (1 to 2 inches in diameter) so that deviations from the true grade do not exceed the diameter of the drain tube. For 5-inch drain tile installed with present ditching machines, small deviations from true grade (0.1 to 0.2 foot) may not seriously affect performance of tile drains especially on steep slopes. For small-size drain tubes installed with the wheel-type (Killefer) mole plow, the mole channel will be very nearly parallel to the soil surface. Thus, for small drains the effect of even small irregularities in the soil surface may be serious. In the low spots along the drain air is trapped when the water level in the drain touches the top of the tube. In small diameter tubes the surface tension of the water along the circumference of the tube acts as a resisting force and hence each air pocket will transmit less force than the preceding one. This effect has been observed in straight capillary tubes and is known as the Jamin effect. The head required to force the water out of the tube is proportional to the number of air traps. In this study an attempt has been made to analyze this effect in perforated drain tubes 1 inch in diameter by embedding the tubes in sand and also in disturbed soil.

The objective of the laboratory investigation was to determine the effect of deviations from true grade on the performance of small perforated drain tubes. The performance was evaluated primarily by measuring the back pressure or head required to cause movement of water through the tube. To a limited extent the effect of the number of deviations from true grade and the amount of deviation (distance $\underline{x}$ in Fig. 24) was determined.

Apparatus and materials

The laboratory tests were made by using a box-shaped tank 10 feet long, 1.3 feet wide, and 1 foot in depth in which three 1-inch type K copper tubes were placed as shown in Figs. 24, 25, and 26. The three copper drain tubes were similar, except for the number of deviations per unit length and the amount of deviation from true grade (see Fig. 27 and Table 5). In bending the copper tubing to the desired shape sand was placed in the tubes and the ends were closed to prevent distortion of the circular cross section. The tubes were shaped by bending against a wooden form of the desired curvature. It was impossible to prevent a slight smashing of the tube, particularly with tube number 2 which had one deviation per foot. Minimum inside diameter for each tube after bending is shown in Table 5. After shaping the tubes,

Table 5

Dimensions of Drain Tubing Used in Laboratory Tank*

Tube no.	Minimum inside diameter at bends	Deviations from true grade (Dimension *x* in Fig. 24)	Number of deviations per foot
	inches	inches	
1	0.91	1.0	0.5
2	0.76	1.0	1.0
3	0.85	1.5	0.5

*Type K copper tubing 1-inch inside diameter; wall thickness 0.085 inches; diameter of perforations 1/8 inch; 12 perforations per foot

they were placed in the tank and supported at each 2-foot interval by small wood blocks which set on the bottom of the tank. Four rows of 1/8-inch perforations (12 per foot),

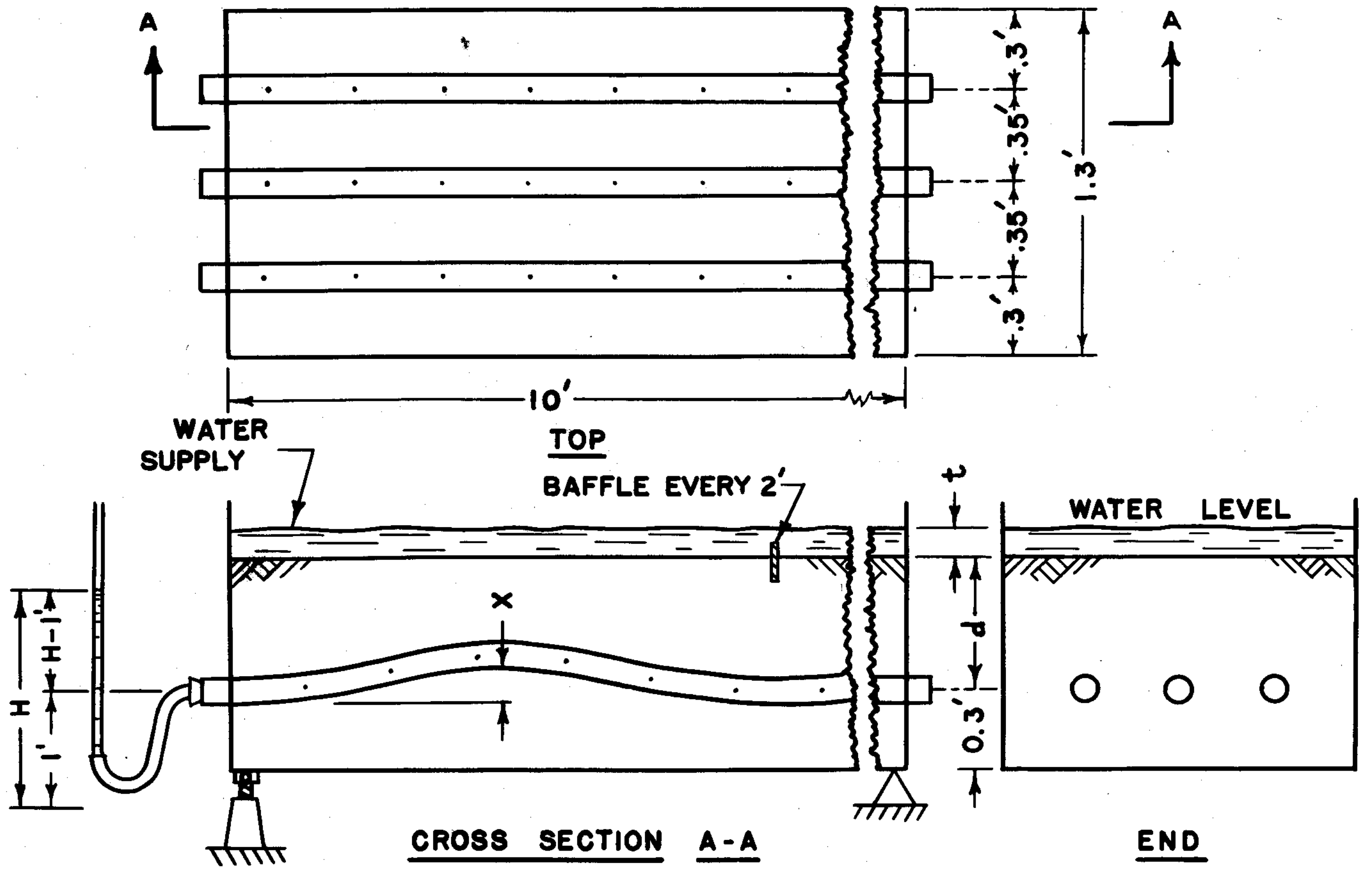


Fig. 24 Dimensions and Details of Laboratory Tank

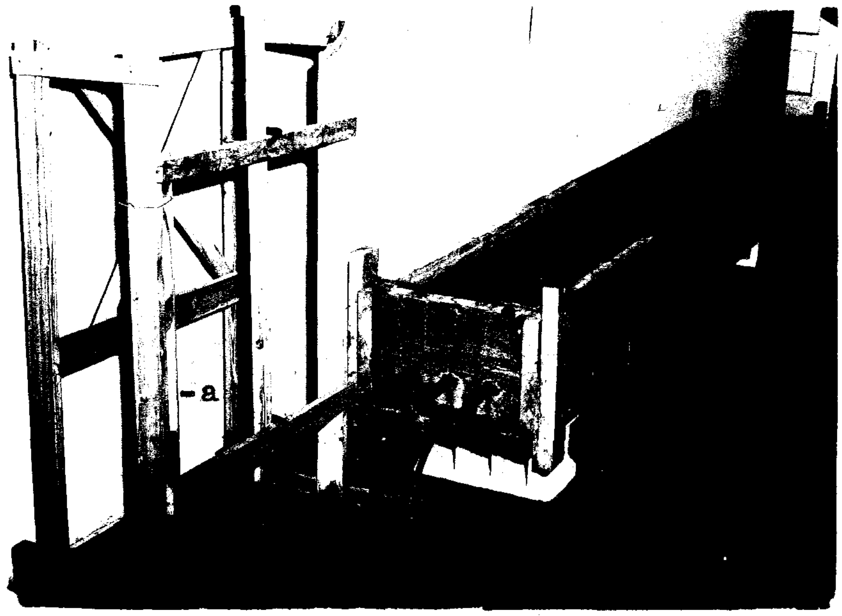
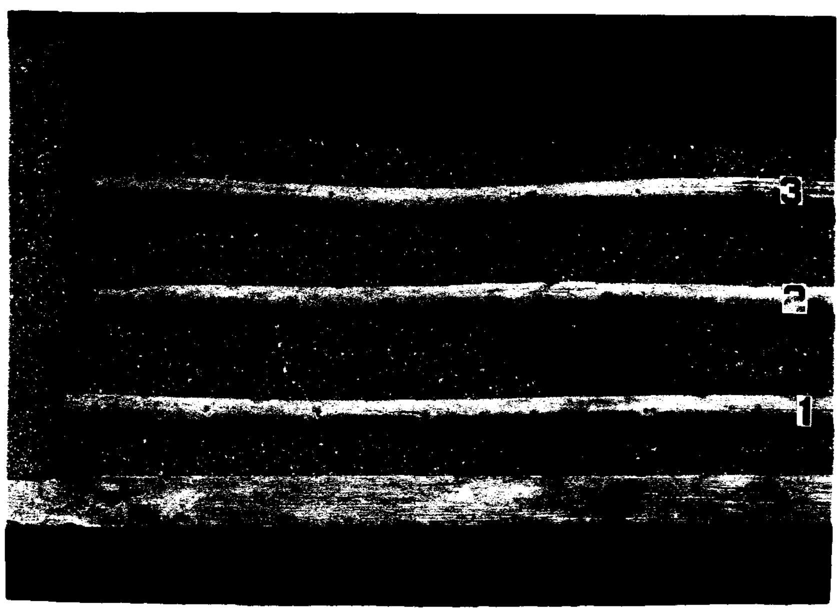


Fig. 25 Laboratory Tank and Apparatus for Measuring Head



(Top View)

Fig.26 Copper Tubes as Placed in Laboratory Tank

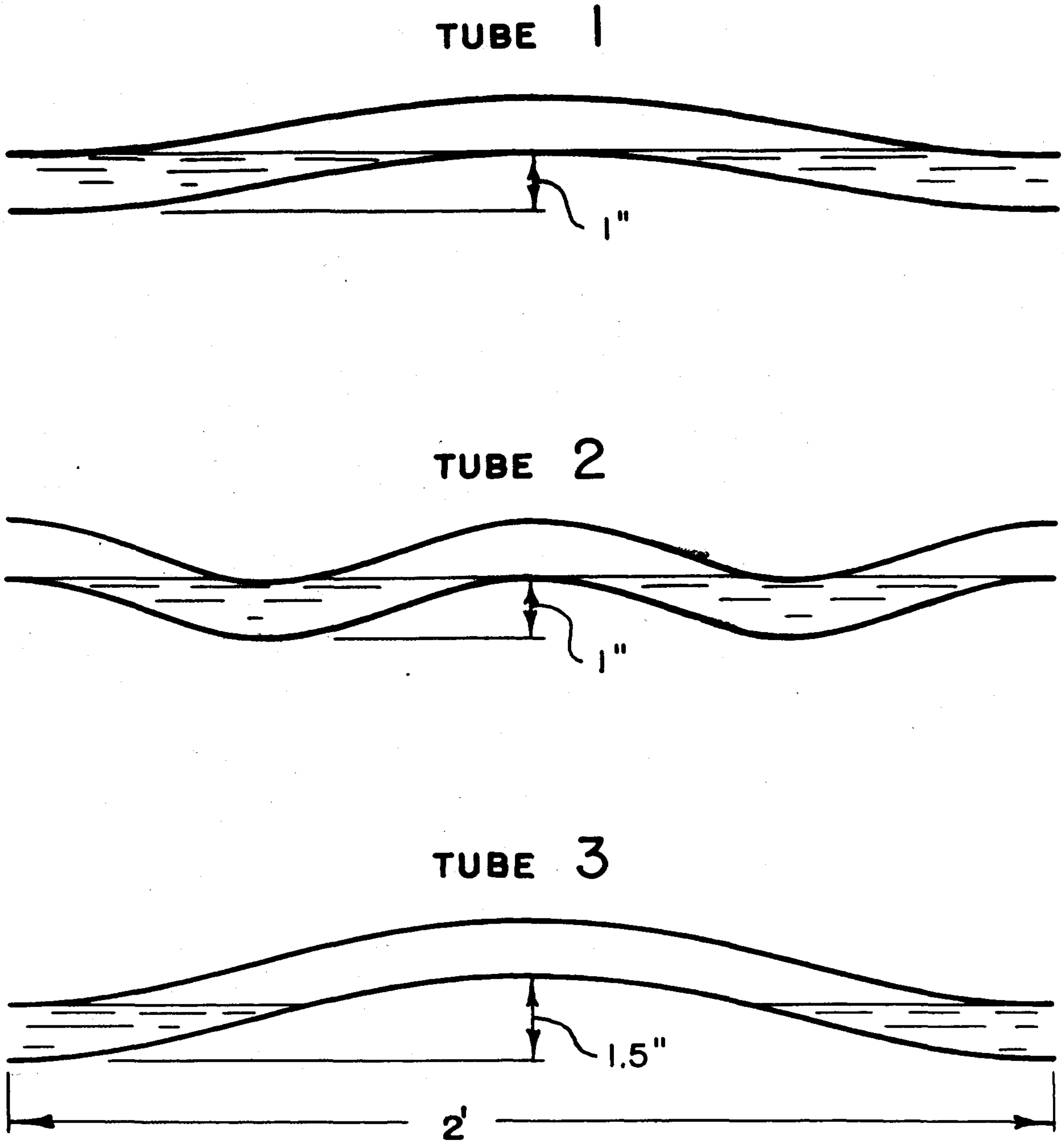


Fig. 27 Longitudinal Cross Section of Copper Tubes

uniformly spaced along the entire length, were drilled in the tubes.

The tank was mounted on a pivot at one end, the other being adjustable in order to provide the desired slope. A small glass tube was used at the upper end to measure the head H. A small reservoir (1-1/2-inch diameter glass tube shown in Fig. 25) connected to each of the tubes at the upper end was used to supply the water for the tests made under nonsaturated conditions. After placing the flow medium (sand or soil) in the tank, baffles were placed at 2-foot intervals along the surface (see Fig. 24) so that a thickness of water t could be maintained above the surface of the flow medium when the tank was inclined.

The sand described in Table 6 was taken from the Des Moines river and had an effective size of 0.38 mm. and a uniformity coefficient of 3.24.

Table 6

Mechanical Analysis of Sand

U. S. sieve number	Size of opening	Per cent passing
	mm.	
200	0.074	0.05
100	0.149	1.36
50	0.297	5.87
40	0.417	13.20
20	0.833	38.40
10	1.981	80.60
---	63.500	100.00

The soil used in the tests was topsoil secured from creek bottom land on the Atomic Research Farm. This soil, taken from the top 8 inches, was an alluvial soil classified as Wabash silt loam. The land had been in corn the previous year and the moisture content at the time the sample was taken was near field capacity. It had a hygroscopic moisture content of 3.3 per cent and a mechanical analysis of 24.4 per cent sand, 55.0 per cent silt, 20.6 per cent 0.005 mm. clay including 14.8 per cent 0.002 mm. clay, as determined by the Bouyoucos hydrometer method.

Method of procedure

Sand was placed in the tank to depth of 0.8 foot under saturated conditions by first placing water in the tank and adding the sand in thin layers. It was then tamped around the tube to insure uniform placement.

Air-dried soil passing a 1/4-inch screen was placed in the tank to a depth of 0.8 foot and tamped slightly to assure uniform packing around the tubes. The soil was saturated slowly and kept saturated for one week before any data were taken. Aggregate analyses were made on samples taken from the top 1 inch of soil. The wet-sieve method as described by Yoder (58) was used.

Saturated conditions. The outflow from each of the tubes was determined under saturated conditions with a thickness of water t on the surface. In making a test, the tube being observed was opened at its lower end, while the other two tubes remained closed. The head H, which was the height of the water column in the glass tube, was measured from the center of the copper tube at its upper end. Regardless of the slope, the center of the tube was taken as the reference point. To eliminate negative values of head and to facilitate statistical analysis the elevation of the center of the tube was arbitrarily assumed to be 1.0 foot. The head H was observed at the time of the outflow measurement. The desired slope was obtained in the tubes by raising the upper end of the tank with a hydraulic jack. With sand as the flow medium the slope varied from zero to 5 per cent and in the case of soil from zero to 4 per cent. For soil the thickness of water on the surface was maintained at 0.10 foot for all slopes, but for sand the thickness of water varied from 0.10 to 0.04 foot for slopes of zero to 5 per cent, respectively. It was not possible to maintain a constant depth of water on the sand due to the high rate of infiltration. Water was supplied at the upper end of the tank and the excess which did not infiltrate into the flow medium was wasted at the lower end. It was possible to control the quantity of water above each baffle by regulating the flow over it. Tap water was used for all tests, the temperature ranging from 53 to 63° F.

<u>Nonsaturated conditions</u>. The head H for nonsaturated conditions was determined after the flow from the tubes

had ceased. The small storage reservoir (a in Fig. 25) supplied water to the upper end of the tube. By raising the water level the head caused movement through the tube. After a few minutes, the water in the reservoir reached a level which was in equilibrium with the frictional forces retarding the movement. At this point the outflow from the tube very nearly stopped and the head H was that due to the Jamin and other effects. Two 1/2-inch diameter observation wells were placed midway between the tubes at the upper end to determine the water level in the soil and to compare this with the head H. The head for nonsaturated conditions was measured only in the case when soil was the flow medium. The high permeability of the sand did not permit such measurements under nonsaturated conditions.

Tubes without perforations. The head for the copper tubes without perforations was determined as previously described for nonsaturated conditions. The quantity of air trapped was calculated by subtracting the quantity of water remaining in the tube from the total volume of the tube. Various quantities of air were trapped by adding different amounts of water to the tube prior to making a test.

Results

Experimental data were obtained for 1-inch perforated tubes embedded in sand under saturated conditions, for tubes placed in topsoil under saturated and nonsaturated conditions and for the same tubes without perforations. Discharge measurements from the tubes during a period of 141 days were compared with aggregate analyses of the soil. Finally, a comparison of the data was made for all tests.

Saturated sand. When sand was used in the laboratory tank the discharge and the head H varied considerably, making it difficult to duplicate results. Although the sand was removed and replaced several times, this did not appear to improve the data. The head H at various slopes for each of the tubes is shown in Table 7 and Fig. 28. The data taken on October 29 were not plotted because it was not in close agreement with other observations. From the data in Fig. 28 tube 1 gave less variation in head than tube 3. When the measurement of head H was less than 1.0 foot, negative head or suction was indicated.

Although the depth of water on the surface of the sand could not be maintained at 0.10 foot for slopes above 2 per cent, this did not appear to affect the straight line relationship as shown in Fig. 28. In all data plotted the points were very nearly on a straight line for a particular series of observations.

The discharge of the tubes increased with an increase in slope, indicating either that 1/8-inch perforations permitted water to enter the tube faster than it could be carried away or that a reduction in the head or back pressure caused the increased flow. From the data in Table 8

Table 7

Head H in Feet for Saturated Sand

Slope in drain tube %	Date of observation (1949) Aug. 15*	Oct. 29	Nov. 5*	Nov. 13*
	Head H in feet			
		TUBE 1		
0	1.29	1.11	1.29	1.27
1	1.20	0.99	1.22	1.20
2	1.13	0.89	1.15	1.13
3	1.06	0.78	1.08	1.05
4	0.99	0.68	1.00	0.97
5	0.94	0.59	0.95	0.92
		TUBE 2		
0	1.23	1.15	1.49	1.46
1	1.13	1.04	1.43	1.39
2	1.05	0.93	1.37	1.32
3	0.97	0.83	1.30	1.26
4	0.88	0.73	1.23	1.19
5	0.79	0.64	1.17	1.13
		TUBE 3		
0	1.48	1.13	1.25	1.18
1	1.40	1.02	1.16	1.08
2	1.34	0.91	1.08	0.99
3	1.28	0.81	1.00	0.90
4	1.22	0.71	0.92	0.80
5	1.16	0.61	0.84	0.71

*Data plotted in Fig. 28

Table 8

Tube Discharge Q for Saturated Sand

Slope in drain tube %	Date of observation (1949) Aug. 15	Oct. 29	Nov. 5	Nov. 13
	Q in $ft^3/ft/day$			
		TUBE 1		
0	20.95	39.25	16.47	16.58
1	21.46	41.90	17.49	17.80
2	22.17	44.54	18.00	18.51
3	23.69	46.98	19.12	19.32
4	24.20	48.61	19.83	19.93
5	24.41	51.46	20.24	20.75
		TUBE 2		
0	25.83	40.88	14.24	14.44
1	27.25	42.81	15.25	15.46
2	29.08	44.14	15.97	16.47
3	30.81	48.81	16.88	17.19
4	32.64	51.46	18.00	18.00
5	34.88	54.10	18.71	18.81
		TUBE 3		
0	14.24	46.93	20.95	22.58
1	14.54	49.42	22.78	23.39
2	15.46	53.90	23.80	24.46
3	16.27	57.15	24.92	26.03
4	16.98	60.20	26.54	27.46
5	17.80	61.22	27.86	28.98

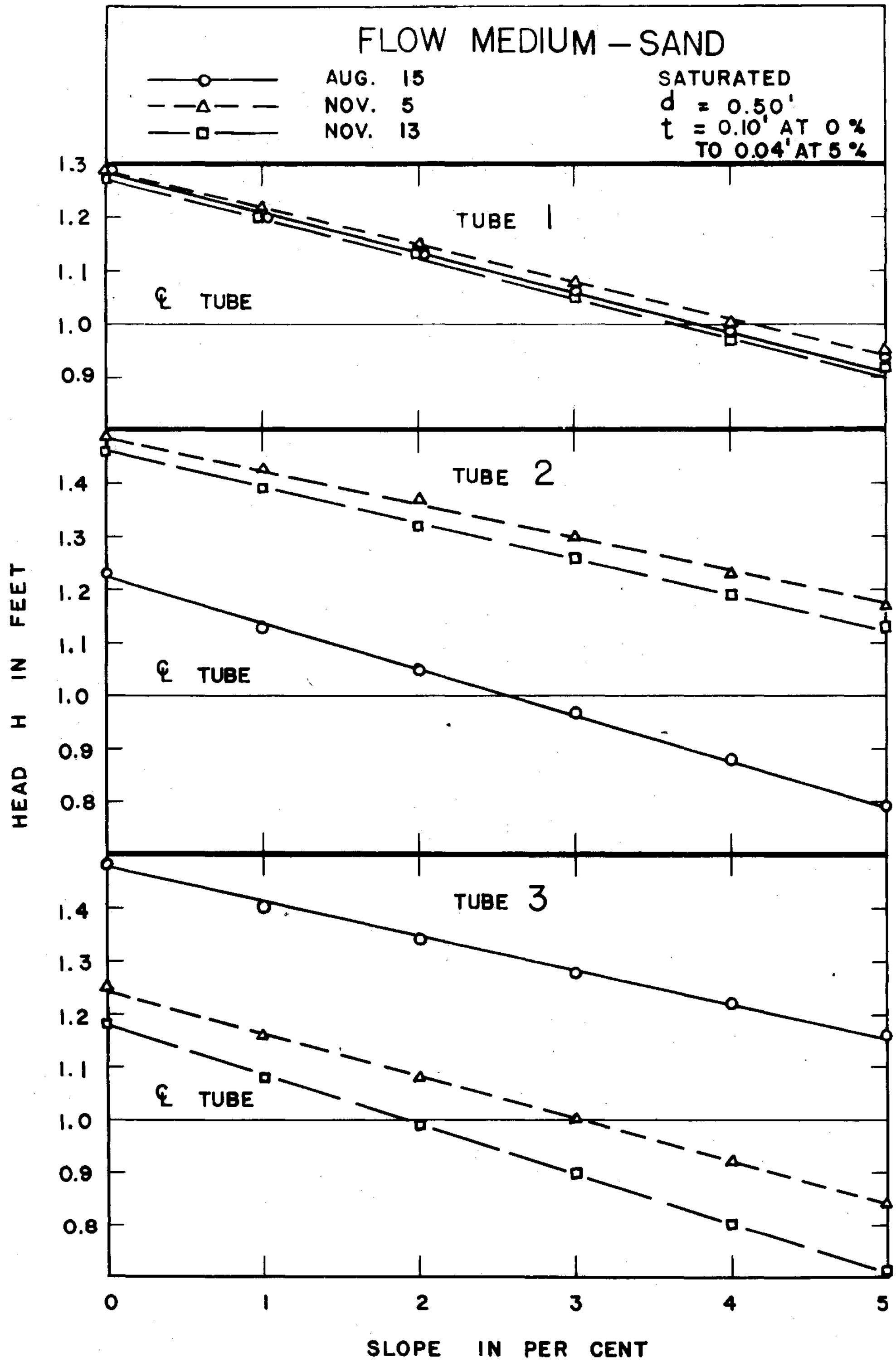


Fig. 28 Head at Various Slopes for Saturated Sand

the discharge was apparently affected neither by the amount of deviation in the tube nor by the number of bends. The discharge varied from about 14 to 47 cubic feet per foot of length per day at zero slope even though every effort was made to secure uniform compaction of the sand. Regression equations (Table 15) for data shown in Table 7 indicated that the head at all slopes from zero to 5 per cent for the three tubes was in increasing order 1, 3, and 2. The _t_ tests for the regression lines were all significant at the 1 per cent level.

Saturated soil. When topsoil was the flow medium in the tank the data shown in Tables 9 and 10 and Fig. 29 were obtained. There was greater variation in head H for tubes 2 and 3 than for tube 1. For tube 1 the head H for slopes up to 4 per cent was very nearly the same as was obtained for the sand. For tubes 2 and 3 a suction was not obtained at the upper end of the tube (head less than 1.00) for slopes up to 4 per cent. However, for tube 1 suction was evident at a slope of about 3 per cent. The data for tube 1 as shown in Fig. 29 did not follow a straight line for the February 22 observation. Apparently, the deviation from the straight line was caused by a decrease in the amount of air trapped in the tube; that is, air escaped and was replaced by water. Regression equations (Table 15) of the data shown in Table 9 indicated that the head H at all points from zero to 4 per cent slope for the three tubes

Table 9

Head H in Feet for Saturated Soil

Slope in drain tube %	2	4*	6	13	20	27	34*	49	76*
	Head H in feet								
				TUBE 1					
0	1.26	1.25	1.25	1.22	1.21	1.22	1.24	1.23	1.22
1	1.19	1.18	1.18	1.14	1.13	1.14	1.16	1.16	1.15
2	1.11	1.10	1.10	1.06	1.04	1.06	1.08	1.07	1.07
3	1.04	1.03	1.03	0.97	0.95	0.97	0.93	0.99	1.00
4	0.95	0.94	0.94	0.85	0.86	0.81	0.84	0.91	0.92
				TUBE 2					
0	1.45	1.45	1.45	1.36	1.35	1.34	1.35	1.36	1.28
1	1.39	1.39	1.38	1.28	1.28	1.27	1.28	1.29	1.23
2	1.34	1.32	1.31	1.21	1.23	1.22	1.22	1.23	1.18
3	1.27	1.26	1.23	1.14	1.15	1.17	1.18	1.18	1.11
4	1.20	1.21	1.19	1.08	1.10	1.11	1.12	1.12	1.05
				TUBE 3					
0	1.47	1.48	1.50	1.37	1.34	1.36	1.38	1.40	1.32
1	1.45	1.45	1.45	1.31	1.26	1.30	1.34	1.37	1.29
2	1.39	1.39	1.38	1.24	1.19	1.27	1.28	1.31	1.22
3	1.33	1.33	1.31	1.17	1.11	1.21	1.25	1.27	1.20
4	1.27	1.27	1.26	1.09	1.03	1.13	1.18	1.20	1.12

*Data plotted in Fig. 29

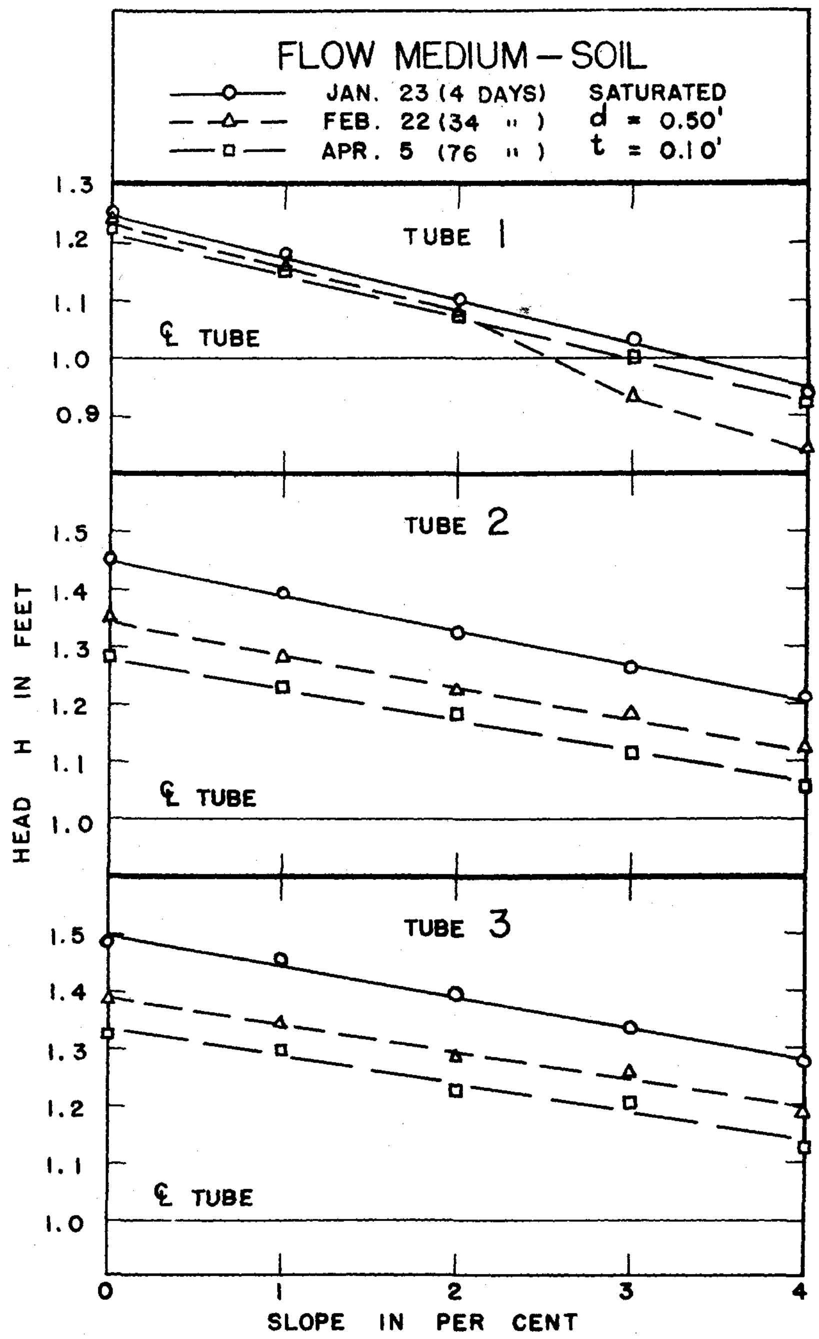


Fig. 29 Head at Various Slopes for Saturated Soil

was in increasing order 1, 2, and 3. As for sand the *t* tests for the regression lines were all significant at the 1 per cent level.

Outflow measurements taken over a period of 141 days indicated considerable variation in discharge which was very nearly the same for all tubes. In general the discharge decreased for the first 97 days and then increased. It was assumed that the variation was caused by a change in the permeability of the soil. An attempt was made to correlate aggregate analyses with the discharge. These analyses are shown in Table 10 and Fig. 30. The correlation coefficient between discharge at zero slope and aggregates greater than 0.25 mm. was 0.508 for tube 1, 0.152 for tube 2, and -0.106 for tube 3. At the 5 per cent level the significant correlation coefficient was 0.666. For tubes 2 and 3 there was practically no correlation, but the data for tube 1 was significant at about the 10 per cent level. Correlation coefficients were of the same magnitude for the discharge at 4 per cent slope. Considering that there was a large variation in the aggregate analysis samples (the samples were taken only at the surface of the soil to prevent disturbing the flow paths) and that there were sampling errors in the discharge measurements, the true correlation coefficient should be better than indicated.

Nonsaturated soil. The head at slopes from zero to 4 per cent was determined for each of the tubes in nonsaturated

Table 10

Aggregate Analysis of Wabash Silt Loam Soil

Time in days after initial wetting	Per cent retained on sieve*					Passing 0.105 mm sieve
	2.0 mm sieve	0.991 mm sieve	0.495 mm sieve	0.250 mm sieve	0.105 mm sieve	
13	5.49	12.43	15.05	19.58	19.15	28.30
20	4.80	12.33	14.23	19.00	21.00	28.64
27	4.59	14.30	16.75	20.85	18.20	25.31
34	3.85	12.30	16.64	20.90	20.95	25.36
49	3.84	12.10	15.38	21.43	22.62	24.63
76	3.50	11.23	14.71	20.70	20.80	29.06
97	3.59	11.03	16.03	21.32	22.21	25.82
132	3.78	12.70	18.90	23.75	19.25	21.62
141	4.59	13.26	18.16	20.55	20.30	23.14

*Each figure an average of two soil samples

soil. The results shown in Table 12 and Fig. 32 more nearly represent actual field conditions than do the tests for saturated soil or sand. The head under nonsaturated conditions was nearly 0.1 foot lower than in saturated soil for each of the three tubes for slopes up to 4 per cent. Data were not taken for tube 1 at a slope of 4 per cent as there was neither suction nor head; that is, the tube behaved as though it were a straight tube. This effect was expected in tube 2 as it had the same deviation from true grade as

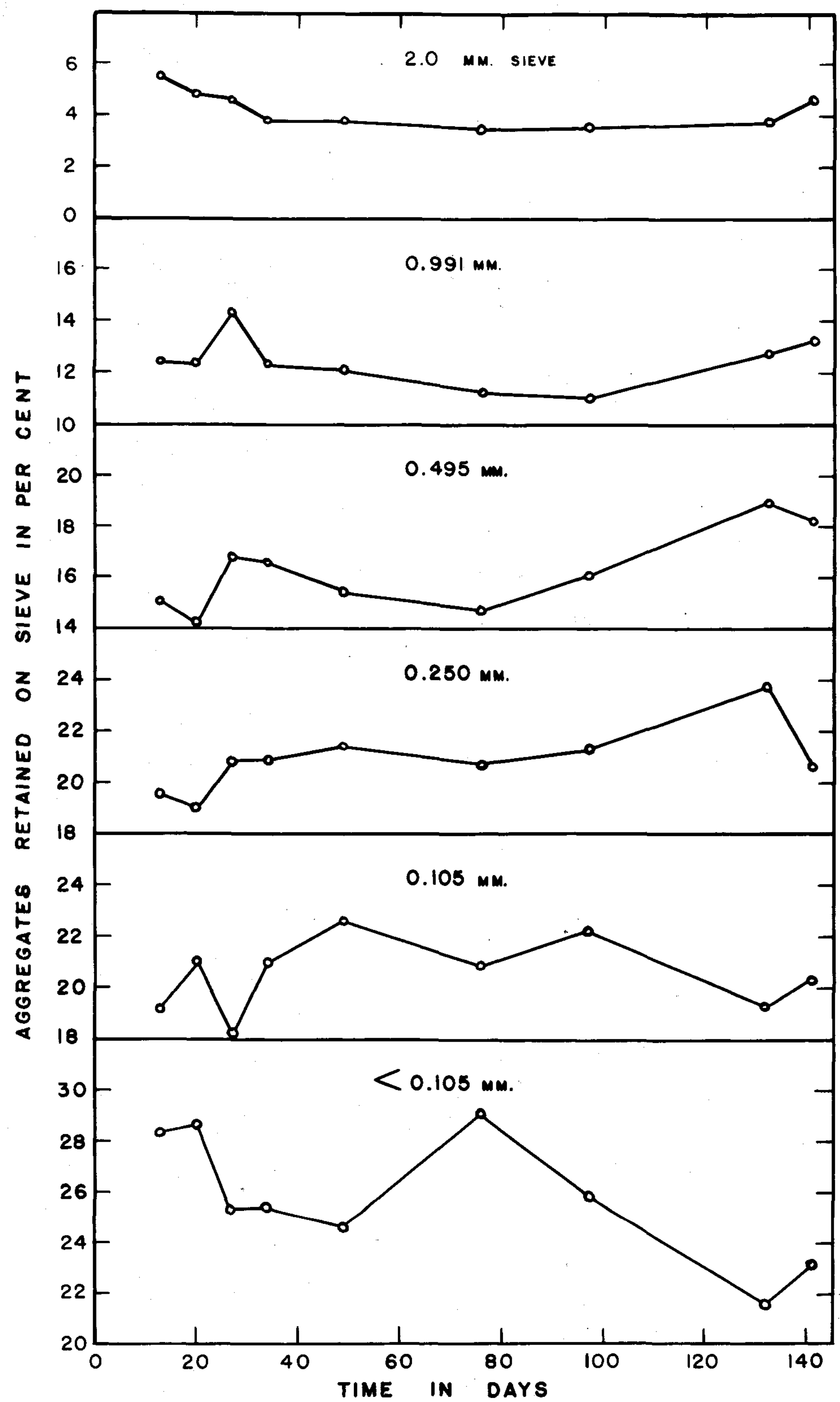


Fig. 30 Aggregate Analysis of Wabash Silt Loam Soil

Table 11

Tube Discharge Q for Saturated Soil

Slope in drain tube %	Time in days after initial wetting 2	4	6	13	20	27	34	49	76	97	132	141
	Q in ft^3/ft/day											
						TUBE 1						
0	4.32	4.48	4.48	2.90	3.16	3.26	3.00	2.60	2.24	1.83	3.05	3.05
1	4.83	4.78	4.73	3.00	3.36	3.36	3.26	2.80	2.49		3.21	
2	4.88	4.98	4.93	3.26	3.56	3.61	3.46	2.95	2.70		3.41	
3	5.13	5.24	5.13	3.51	3.72	3.92	3.72	3.15	2.85		3.56	
4	5.24	5.54	5.29	3.97	4.02	4.22	3.97	3.36	2.95	2.29	3.82	4.12
						TUBE 2						
0	3.26	3.31	3.36	2.39	2.59	2.70	2.44	2.04	2.01	1.58	2.09	2.24
1	3.51	3.61	3.61	2.59	2.70	2.80	2.65	2.24	2.24		2.49	
2	3.61	3.82	3.87	2.85	2.80	3.00	2.80	2.44	2.39		2.59	
3	3.82	4.02	4.12	3.11	3.00	3.11	3.00	2.59	2.54		2.80	
4	3.92	4.07	4.12	3.31	3.15	3.31	3.10	2.75	2.49	2.21	2.80	2.95
						TUBE 3						
0	3.66	3.81	3.87	2.44	3.21	2.90	2.65	2.19	1.91	1.43	2.09	2.14
1	3.77	3.97	4.12	2.65	3.41	3.05	2.80	2.24	2.09		2.44	
2	4.12	4.27	4.48	3.05	3.66	3.21	2.95	2.49	2.24		2.65	
3	4.27	4.42	4.68	3.31	3.92	3.46	3.05	2.59	2.34		2.85	
4	4.48	4.62	4.88	3.56	4.12	3.72	3.31	2.80	2.49	2.03	2.90	2.95

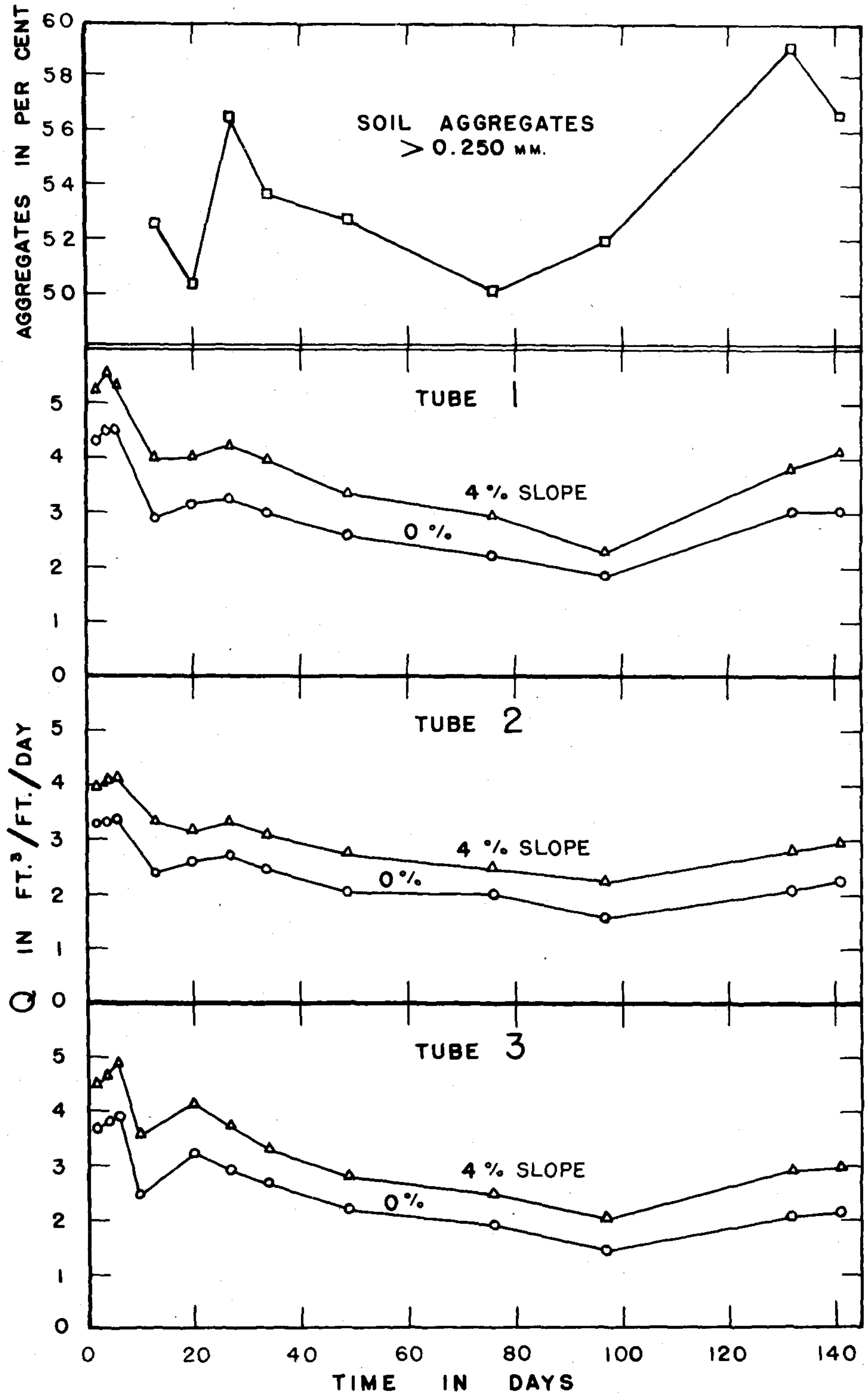


Fig. 31 Effect of Soil Aggregation on Discharge

Table 12

Head H in Feet for Nonsaturated Soil

Slope in drain tube	Time in days after initial wetting						
	28*	35	42*	48	76	98	132*
%	Head H in feet						
			TUBE 1				
0	1.17	1.17	1.17	1.17	1.16	1.16	1.12
1	1.08	1.07	1.09	1.07	1.05	1.04	1.03
2	0.97	0.98	1.00	0.96	0.95	0.94	0.96
3	0.92	0.92	0.93	0.90	0.90	0.92	0.87
4**							
			TUBE 2				
0	1.30	1.30	1.25	1.33	1.26	1.30	1.24
1	1.27	1.21	1.19	1.23	1.17	1.19	1.16
2	1.19	1.12	1.11	1.12	1.08	1.12	1.05
3	1.09	1.08	1.03	1.06	0.99	1.08	0.98
4	1.08	0.98	0.98	0.99	0.91	0.96	0.90
			TUBE 3				
0	1.33	1.36	1.36	1.37	1.35	1.30	1.26
1	1.31	1.33	1.31	1.26	1.27	1.21	1.20
2	1.26	1.16	1.28	1.21	1.21	1.16	1.12
3	1.19	1.11	1.21	1.13	1.15	1.09	1.05
4	1.15	1.10	1.17	1.06	0.82?	1.08	0.98

*Data plotted in Fig. 32

**At 4% slope drain tube had neither suction nor head

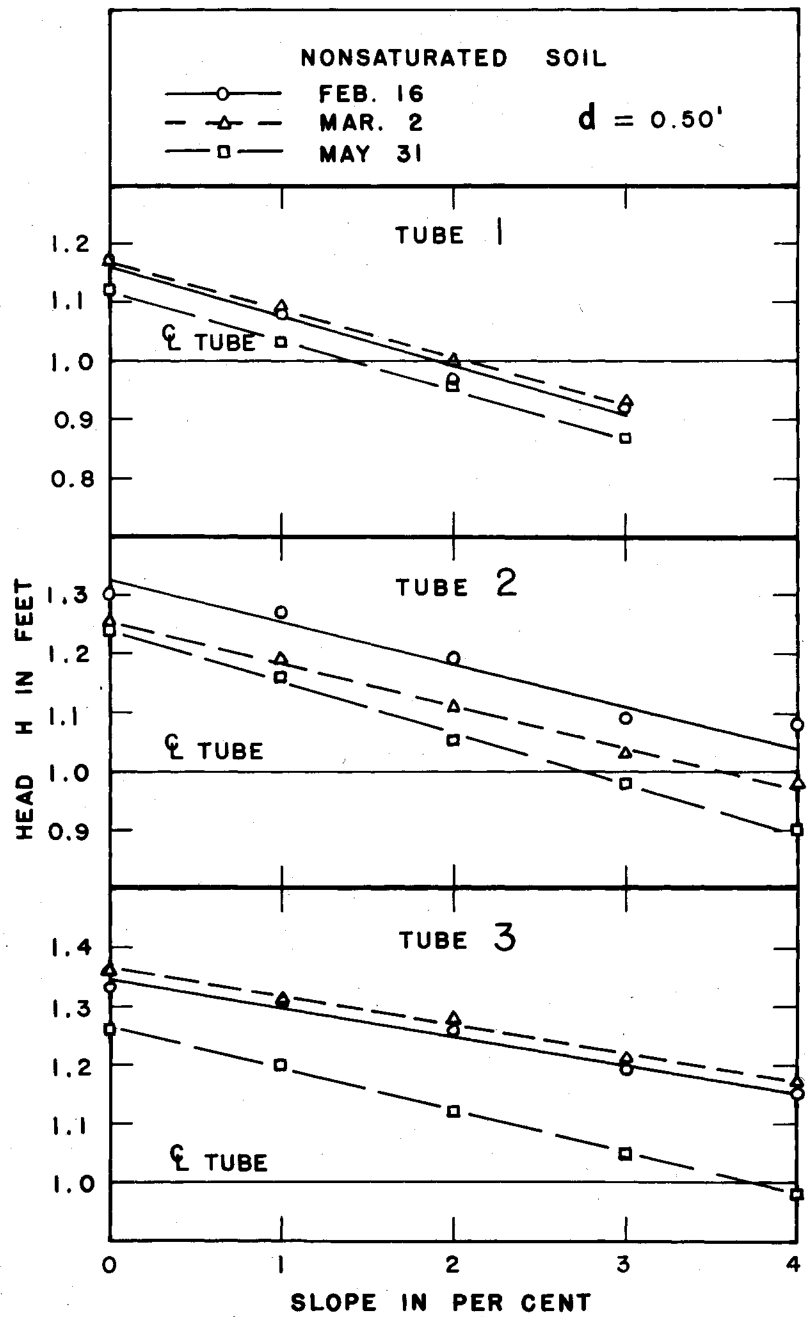


Fig. 32 Head at Various Slopes for Nonsaturated Soil

tube 1, but due to a slightly smaller inside diameter (Table 5) caused by bending the tube in shorter curves, the tube did not perform the same. Some of the variation in data for tube 2 at 4 per cent slope may have been caused by the smaller diameter at the bends.

Regression equations (Table 15) of all the data shown in Table 12 indicated that the head for the three tubes was in increasing order 1, 2, and 3 which was the same as found for saturated soil. All _t_ tests for the regression lines were again significant at the 1 per cent level (Table 15).

Tubes without perforations. The head at slopes from zero to 4 per cent was determined for each of the tubes after sealing the perforations. The data are shown in Table 13 and Fig. 33. Although tubes without perforations cannot be used in drainage work, it was considered desirable to compare their performance with perforated tubes. As before, the head at slopes from zero to 4 per cent plotted as a straight line and there was considerable variation for different observations. At the beginning of the tests various quantities of water were placed in the tubes in order to take measurements for varying amounts of entrapped air. As shown in Table 13 the head H was influenced by the amount of water in the tube at the start of a test.

Since the volume of air in the tubes affected the head, additional tests were made at zero, 2, and 3 per cent slopes. The data are shown in Table 14 and Fig. 34. As

Table 13

Head H in Feet for Tubes Without Perforations

Slope in drain tube	Observation 1	2	3	4
	Water in tube at start of test			
	Unknown*	Full	None	None
%	Head H in feet			
		TUBE 1		
0	1.14	1.07	1.07	1.08
1	1.06	0.97	0.98	1.00
2	0.97	0.88	0.89	0.92
3	0.87	0.79	0.79	0.85
		TUBE 2		
0	1.16	1.08	1.13	1.18
1	1.06	0.98	1.01	1.10
2	0.96	0.88	0.92	1.00
3	0.86	0.78	0.83	0.90
4	0.76	0.72	0.72	0.81
		TUBE 3		
0	1.45	1.13	1.35	1.36
1	1.37	1.03	1.27	1.27
2	1.30	0.94	1.19	1.19
3	1.23	0.84	1.12	1.11
4	1.16	0.75	1.03	1.02

*Same method for starting test as for nonsaturated soil

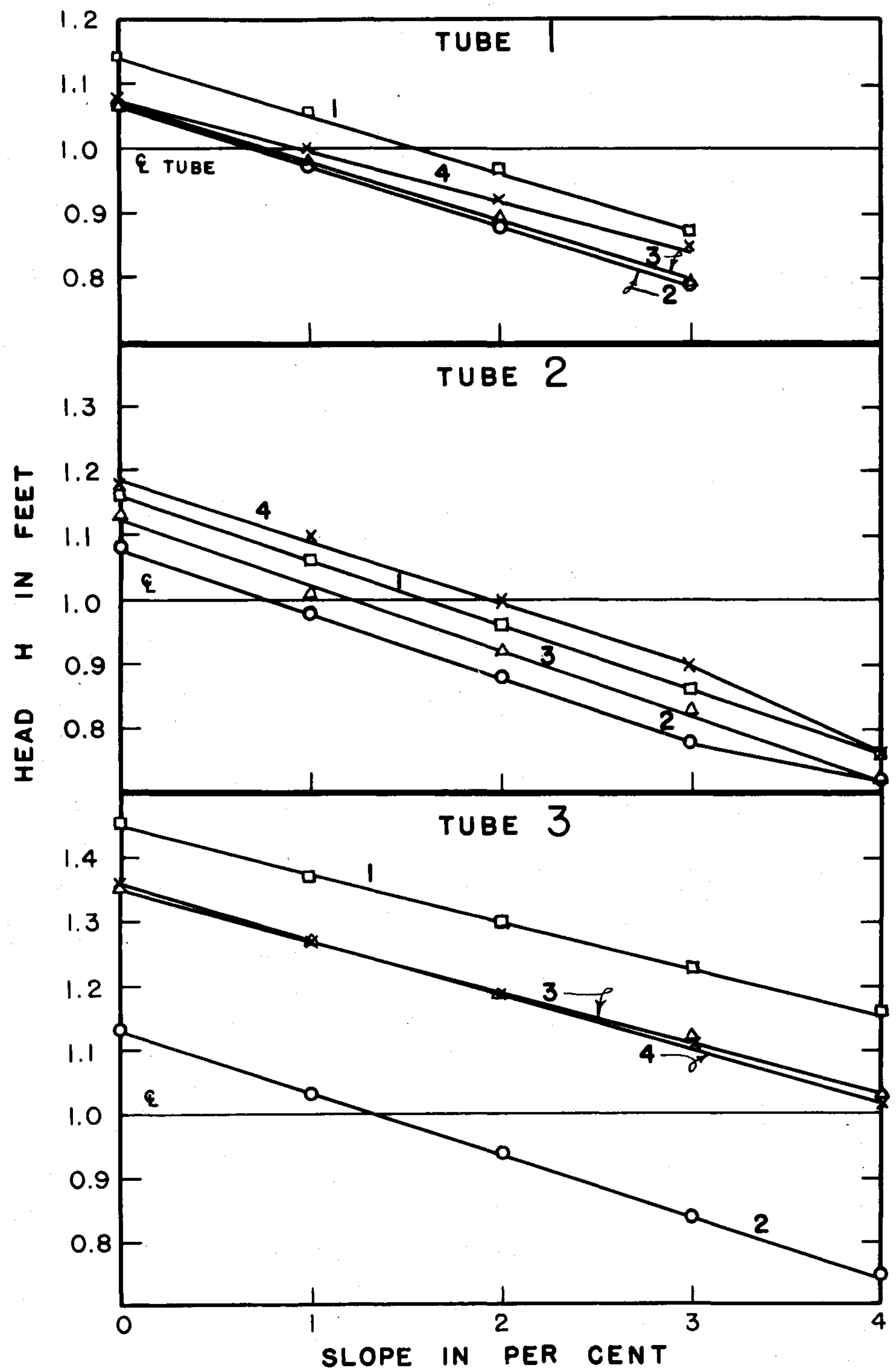


Fig. 33 Head at Various Slopes for Copper Tubes Without Perforations

Table 14

Volume of Air V in Tubes and Head H

			Slope in per cent					
	0			2			3	
Test no.	V* %	H ft.	Test no.	V* %	H ft.	Test no.	V* %	H ft.
				TUBE 1				
511	18.5	1.07	583	52.5	0.94	610	59.1	0.88
580	18.5	1.07	586	52.5	0.95	601	56.2	0.86
571	26.8	1.07	589	43.5	0.93	607	59.7	0.82
550	34.5	1.08	592	20.8	0.89	613	41.7	0.82
548	34.5	1.08	595	20.3	0.89	604	22.7	0.80
546	49.4	1.09	598	45.3	0.92			
574	50.6	1.09						
577	50.6	1.10						
	t = 9.486**			t = 8.18**			t = 1.82	
							$t_{.20}$ = 1.64	
				TUBE 2				
512	13.0	1.08	584	50.7	1.01	608	52.4	0.90
581	12.4	1.09	587	49.5	0.97	611	51.3	0.86
551	27.7	1.10	590	51.8	1.03	602	49.5	0.85
572	24.1	1.11	593	14.7	0.90	614	43.7	0.84
575	47.0	1.19	596	14.7	0.89	605	15.9	0.78
578	45.8	1.19	599	50.1	0.97			
	t = 7.23**			t = 4.90**			t = 4.04	
							$t_{.05}$ = 3.18	
				TUBE 3				
513	17.1	1.13	585	51.8	1.23	612	51.3	1.07
582	16.5	1.14	588	51.8	1.21	609	51.8	1.06
573	29.4	1.18	591	55.3	1.29	603	50.1	1.04
552	32.3	1.21	594	17.7	0.95	615	44.2	1.01
547	49.4	1.36	597	17.7	0.95	606	19.4	0.85
576	50.6	1.36	600	48.3	1.11			
549	50.6	1.37						
579	50.0	1.38						
	t = 14.28**			t = 6.82**			t = 61.70**	

*Volume of air in tube as per cent of totsl volume of tube

**Significant at the 1% level

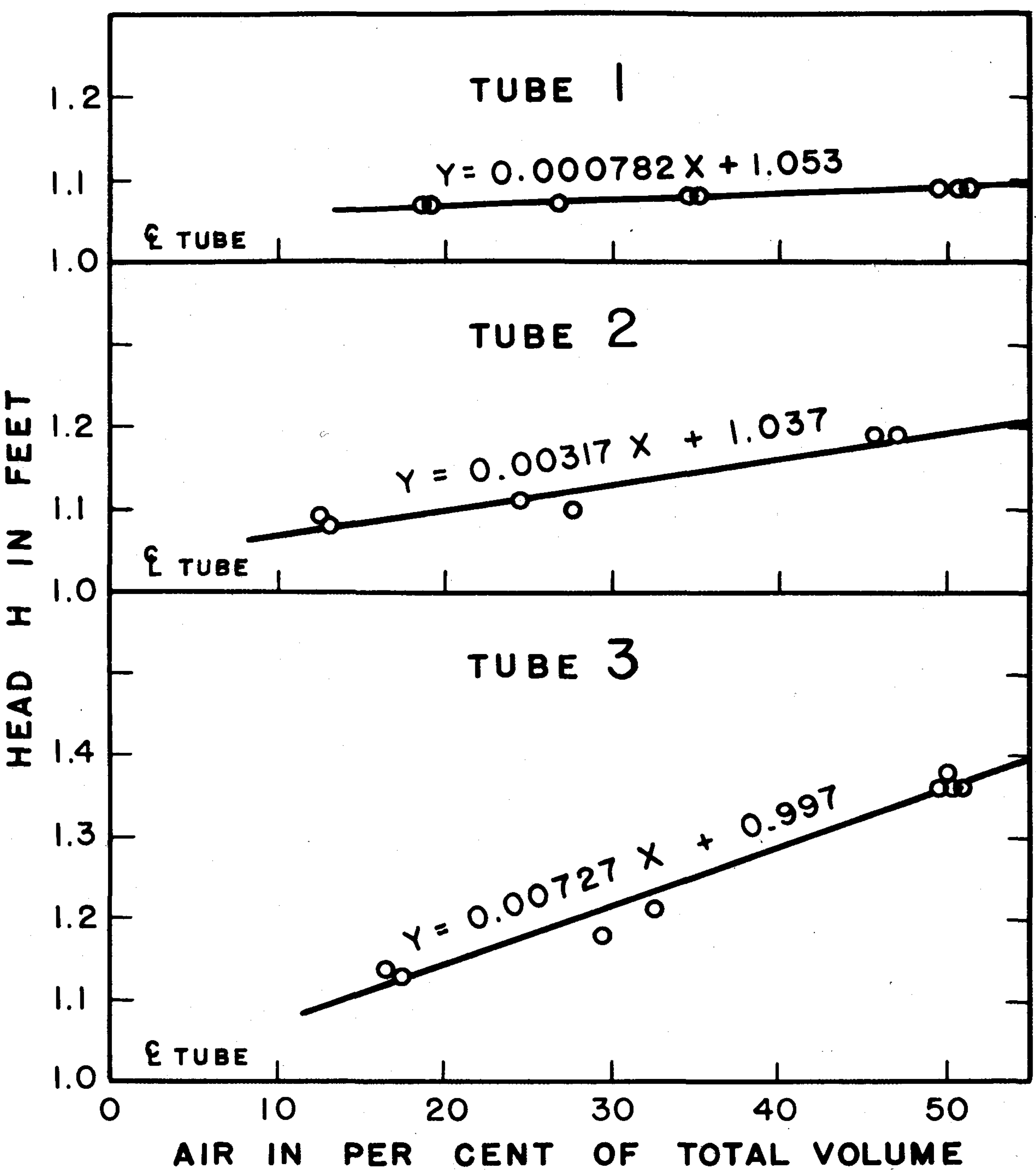


Fig. 34 Effect of Air in Tube on Head at Zero Slope

the plot of the head was a straight line for slopes up to 4 per cent, the per cent of air was not measured at other slopes. The regression equations for zero slope are plotted in Fig. 34. The slope of the regression lines for the tubes was in increasing order 1, 2, and 3 which explains why more variation in head was obtained for different observations in the case of tube 3 than for tube 1. The regression line for tube 1 was very nearly horizontal indicating that a large variation in the volume of air had a small effect on the head. With two exceptions the <u>t</u> tests for the regression lines given in Table 14 were significant at the 1 per cent level.

<u>Comparison of tests</u>. A summary of the effect of deviations from true grade in 1-inch drain tubes is given in Table 15 and Fig. 35. Since the data for sand were not consistent, few observations were made and they were not included in Fig. 35. In all cases the head for perforated tubes in saturated and nonsaturated soil was greater than the head for the tubes without perforations, and the head was greater in saturated than in nonsaturated soil.

Perforated drain tubes which have deviations from true grade and are placed in nonsaturated soil perform very similarly to tubes without perforations. Although the regression lines for this data in Fig. 35 show that the head is approximately 0.02 to 0.2 foot higher for nonsaturated soil than for the tubes without perforations, this difference may be caused by the variation in the percentage of air trapped in the tube (Fig. 34).

Table 15

Summary of the Statistical Analysis for Head and Slope in One-Inch Drain Tubes

Treatment	Tube no.	No. of observations n	Regression equation Y = Head H X = Slope in %	t	$t_{.01}$
Saturated sand	1	24	Y = 1.23 - 0.079 X	5.41	2.819
(River sand)	2	24	Y = 1.33 - 0.080 X	3.38	2.819
	3	24	Y = 1.25 - 0.085 X	3.99	2.819
Saturated soil	1	45	Y = 1.24 - 0.085 X	25.02	2.696
(Wabash silt loam)	2	45	Y = 1.37 - 0.061 X	10.47	2.696
	3	45	Y = 1.41 - 0.058 X	7.42	2.696
Nonsaturated soil	1	28	Y = 1.15 - 0.085 X	22.31	2.779
(Wabash silt loam)	2	35	Y = 1.28 - 0.078 X	15.25	2.724
	3	35	Y = 1.34 - 0.070 X	8.70	2.724
Tubes without	1	16	Y = 1.09 - 0.088 X	10.92	2.977
perforations	2	20	Y = 1.14 - 0.097 X	13.76	2.878
	3	20	Y = 1.32 - 0.083 X	3.69	2.878

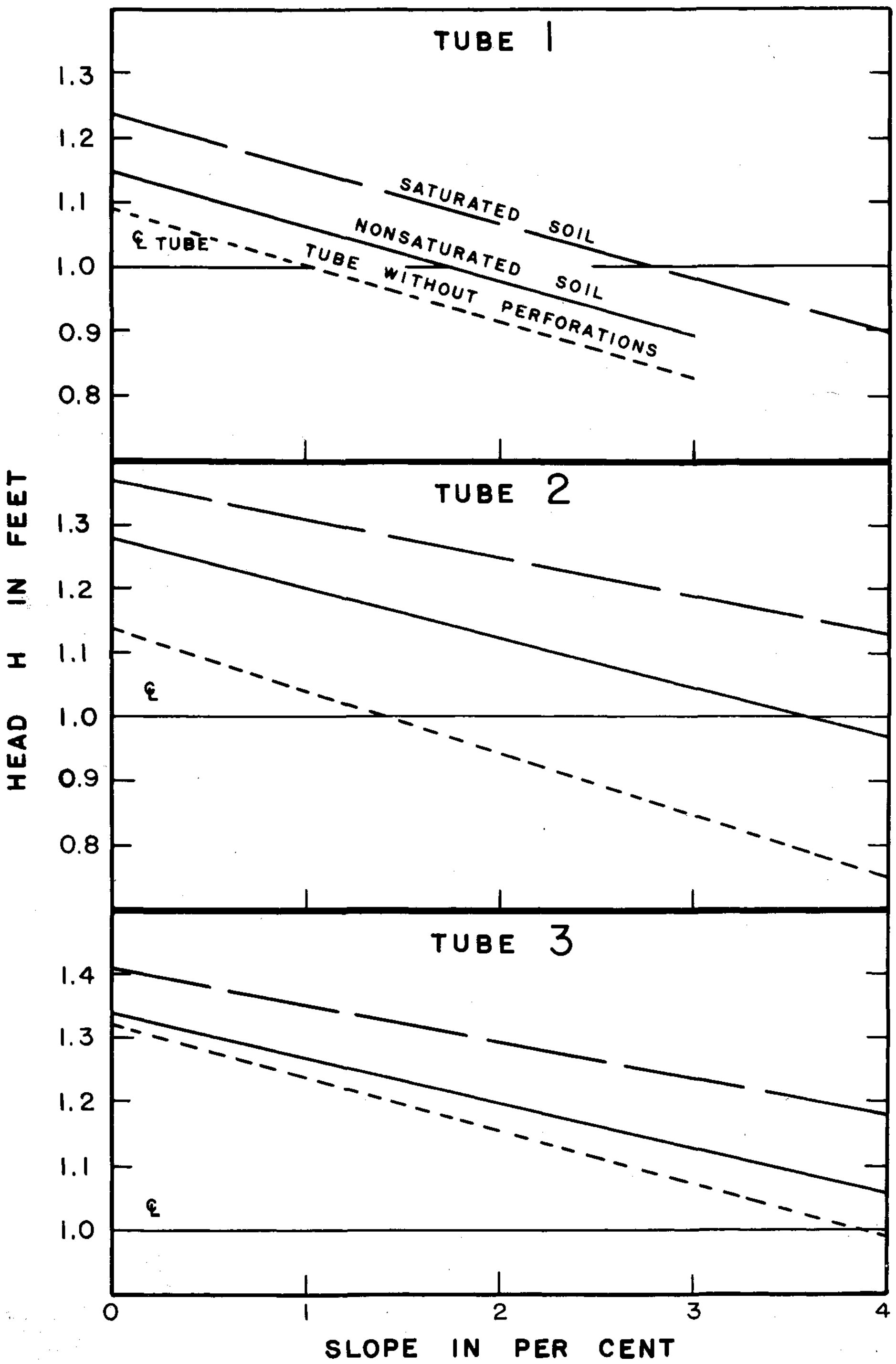


Fig. 35 Regression Lines for Head Versus Slope for All Data

The difference in head between saturated-soil and nonsaturated-soil regression lines may be caused by the percolation of water from the soil into the tube causing an apparent increase in the head. However, some of this difference may be caused by variation in the percentage of air in the tube.

For tubes 1, 2, and 3 at zero slope a correlation coefficient between discharge and head for saturated soil was obtained which was very nearly significant at the 1 per cent level. Hence, for saturated soil the higher the discharge from the 1-inch tubes the higher the head. It should be remembered that if the tube were completely filled with water, siphon action would move the water from one deviation to the next and a low head would result. In some observations this condition was very nearly the case. Since it was not possible to check the percentage of air in perforated tubes, its effect on the head was not known.

Although the data for the regression lines in Fig. 35 were significant, the variation in head was considerable for different sets of observations. With tubes in nonsaturated soil drainage was not reduced (head greater than 1.00) by deviations from true grade of 1 inch with a slope greater than 1.7 per cent. Likewise, for saturated conditions the head was not important for slopes above 3 per cent. In nonsaturated soil tube 2, which had twice as many deviations as tube 1 and was slightly smaller in inside diameter due to bending, reduced drainage for slope less than 3-1/2 per cent. However, under saturated conditions a head of 0.13 foot occurred at 4 per cent slope. In non-saturated soil for tube 3, which was the same as tube 1 except the deviations were 1-1/2 inches, drainage was reduced at slopes greater than about 5 per cent. An increase in the deviations from 1 to 1-1/2 inches resulted in a greater increase in the water table than an increase in the total number of deviations from 5 to 10. For the above conditions the amount of deviation from true grade was more important than the number of deviations.

Some tests made with 1/2-inch tubes showed that 3 deviations from true grade at zero slope gave the same head as tube 3 (1-inch diameter) which had 5 deviations. Indications are that for larger diameters than 1 inch the effect of deviations would be less than shown in these tests. Under field conditions deviations from true grade are likely to occur less frequently than those studied in the laboratory; therefore, for deviations of 1 and 1-1/2 inches the results represent the most severe conditions.

Although the correlation coefficient between discharge and the total percentage soil aggregates greater than 0.25 mm. was not significant, there were indications that aggregation had considerable effect on permeability. If the data presented in Fig. 31 can be applied to field conditions, the flow through the backfill over tile drains will not decrease within a period of 4 or 5 months. The high organic matter content in the topsoil used in the tests apparently affected total aggregation and hence the discharge.

Stability of Perforated Flexible Tubes in Mole Drains

Field investigations were conducted on the stability of perforated flexible drain tubes in mole channels. Polyethylene tubing was used for all field studies because it was economical, possessed suitable physical and chemical properties, and was available commercially. Compared to drain tile of the same size, polyethylene tubing was relatively expensive, but by using smaller diameters and by installing the tubing with a mole plow the cost may be reduced. The cost of tubing was proportional to amount of material per unit length; that is, for a given diameter the cost varied directly with the wall thickness.

The objectives of the field investigations were to determine the stability of perforated flexible tubes in mole drains, to determine the effect of mole channel diameter on stability, to evaluate the effect of different soils on stability, to observe the durability of polyethylene in soil, and to observe the performance of perforated tubes under normal conditions.

Apparatus and materials

All field installations were made with a No. 32 John Deere Killefer mole plow which was used for previous mole drainage work as reported by Gattis (13) and Schwab (45). The original mole plow was modified by adding 6 inches of length to the mole blade and by replacing the rectangular mole point with a 3-inch diameter cylinder tapered on one end. For mole plug diameters less than 2-1/2 inches a home-made blade 1/2-inch thick and 8 inches wide was provided.

The plastic tubes were attached to the mole plow by two methods illustrated in Figs. 36 and 37. Both arrangements were satisfactory. In Fig. 36 a tapered wooden plug and an adjustable hose clamp were used, while in Fig. 37 a metal tube was attached to the plastic tubing by sheet metal screws and to the metal end plug by a machine bolt. With the end plug illustrated in Fig. 37 it was possible to disconnect the tube without raising the plow. The bolt was unscrewed by reaching into a post hole dug directly over the end plug.

Apparatus was devised to measure the inside diameter of plastic tubes in order to determine the degree of failure. For 3- and 4-inch diameter tubes an electrical resistance caliper was designed and constructed as shown in Figs. 38 and 39. The apparatus shown in Fig. 39 consisted of a 5,000-milliohm volume control unit mounted on the

Fig. 36 Method of Attaching Tube to Mole Plug Using Wooden Plug and Hose Clamp

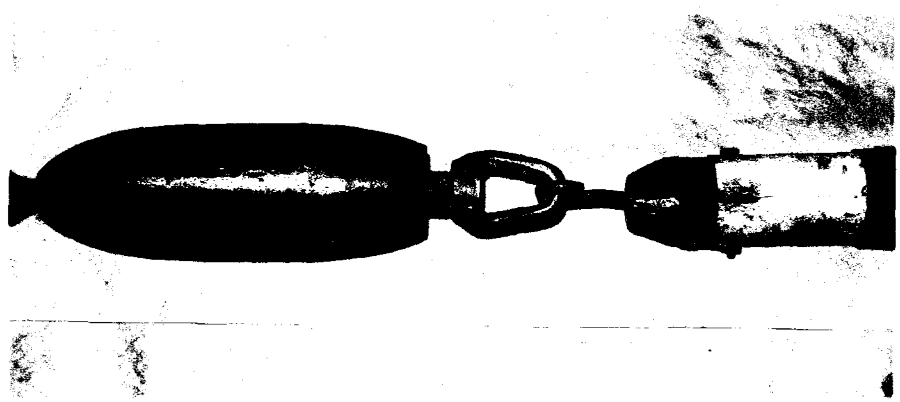


Fig. 37 Method of Attaching Tube to Mole Plug Using Metal Plug and Sheet Metal Tube

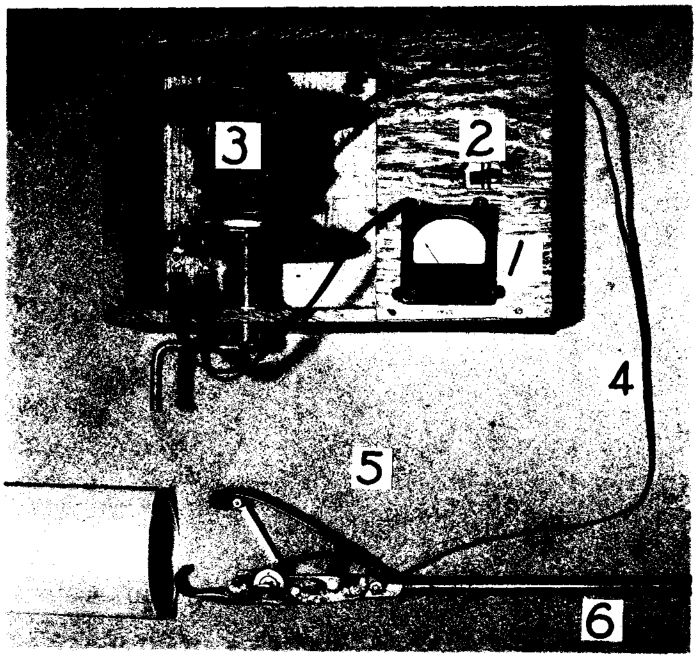


Fig. 38 Electrical Resistance Caliper and Apparatus for Measuring 3- and 4-Inch Diameter Tubes

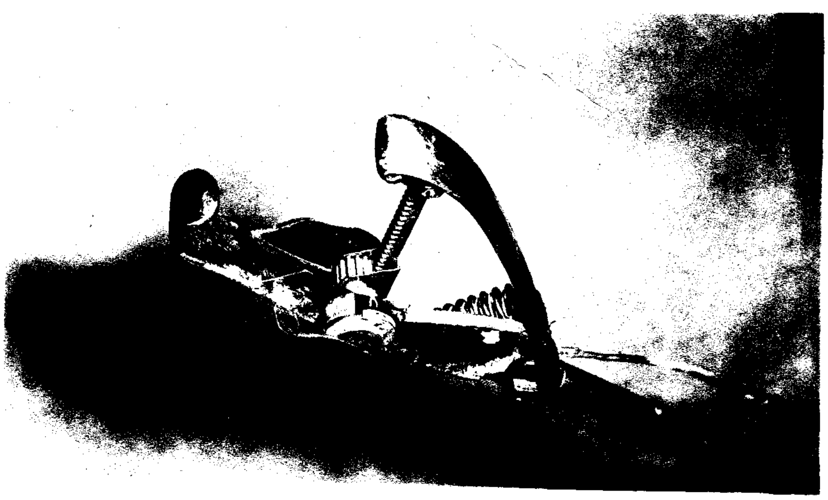


Fig. 39 Close-Up View of Electrical Resistance Caliper

fixed arm of the caliper, a gear wheel and rack to turn the volume control unit, and a movable arm on the caliper to which was attached the rack. A coil spring was used to keep the movable arm of the caliper in the open position. A 1.4-volt dry cell was connected in the circuit as shown in Fig. 40, the current being measured with a milliammeter (1) as shown in Fig. 38. The caliper was calibrated with the ammeter readings for the 3- and 4-inch diameter tubes as shown in Table 16 and Fig. 41. When changing from 4- to 3-inch diameters and vice versa it was necessary to change the relative position of the rack and the gear wheel on the volume control by moving the rack two teeth one way or the other. The 3-inch measurement on the caliper was arbitrarily chosen at 50 milliamps and the 4-inch diameter reading at 35 milliamps. In Fig. 38 the caliper shown in operating condition includes the milliammeter (1), a two-way switch (2) which permits the use of two scales on the ammeter, a 50-foot reel of wire (3), a 2-wire connecting cord (4), the electrical resistance caliper (5) as well as several 4-foot lengths of detachable 1/2-inch steel tubing (6 and 7) which were connected to the caliper for measurement of distances up to 36 feet from the end of the tube.

Since the electrical resistance caliper could not be fitted into tubes 2 inches in diameter or less, other measuring devices were employed. For 1-1/2- and 2-inch diameter tubes steel eye bolts as shown in Fig. 42 were used.

Table 16

Calibration of Electrical Resistance Caliper

Scale on ammeter L 0-25 M 0-250	Ammeter reading	Caliper diameter
	Milliamps	Inches
	For 3-inch diameter tubes	
M	50	3.0
M	35	3.0
M	30	2.9
L	20	2.9
L	14	2.8
L	11	2.8
L	8.0	2.75
L	6.0	2.7
L	5.0	2.6
L	4.8	2.6
L	3.7	2.5
L	3.7	2.5
L	3.2	2.4
L	3.1	2.3
L	3.0	2.25
L	2.8	2.2
L	2.7	2.0
L	2.7	2.0
	For 4-inch diameter tubes	
M	35	4.0
L	11	3.9
L	5.0	3.8
L	2.8	3.7
L	2.3	3.6
L	2.0	3.5
L	1.8	3.4
L	1.6	3.3
L	1.5	3.2
L	1.3	3.1
L	1.2	3.0
L	1.0	2.8

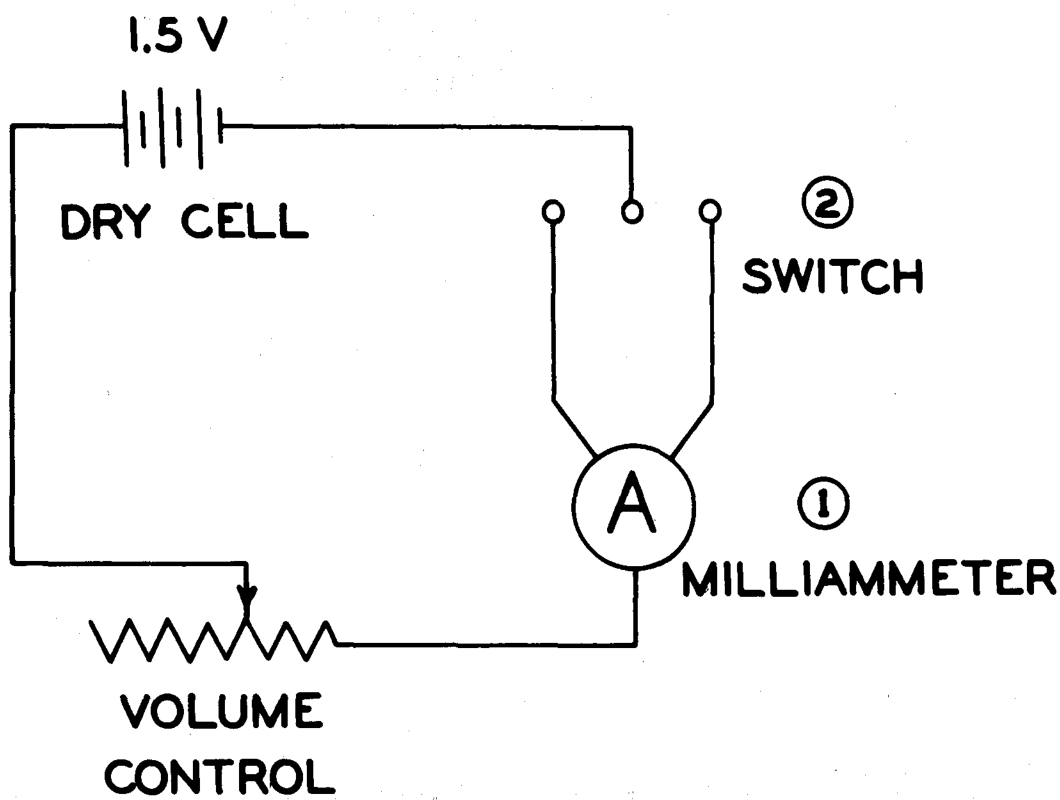


Fig. 40 Wiring Diagram for Electrical Resistance Caliper

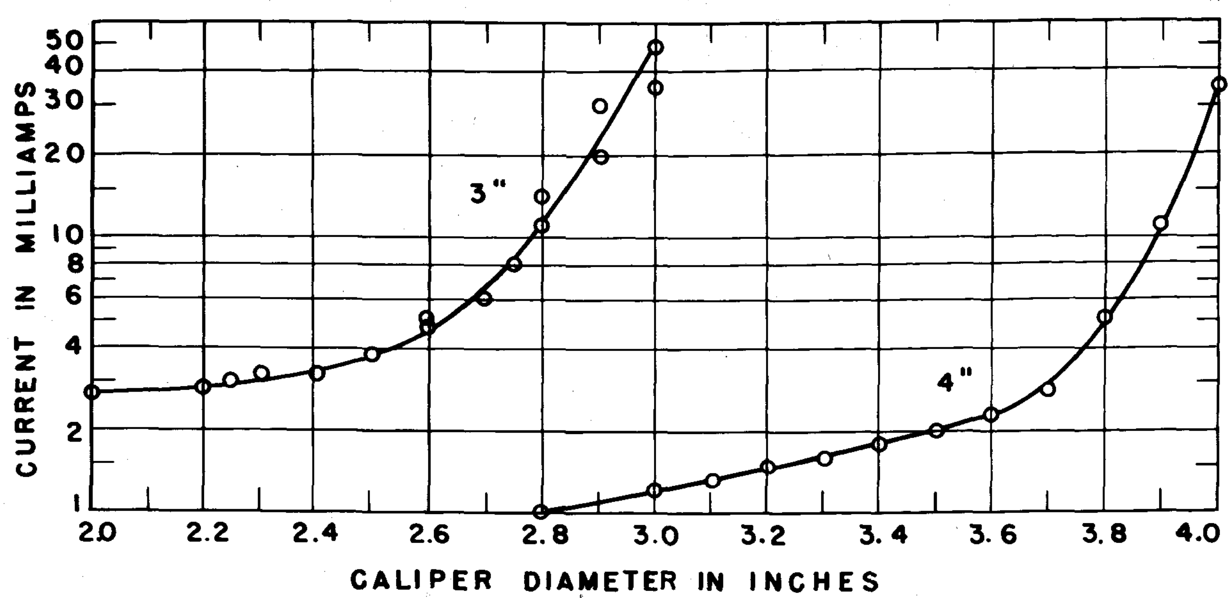


Fig. 41 Calibration Curves for Electrical Resistance Caliper

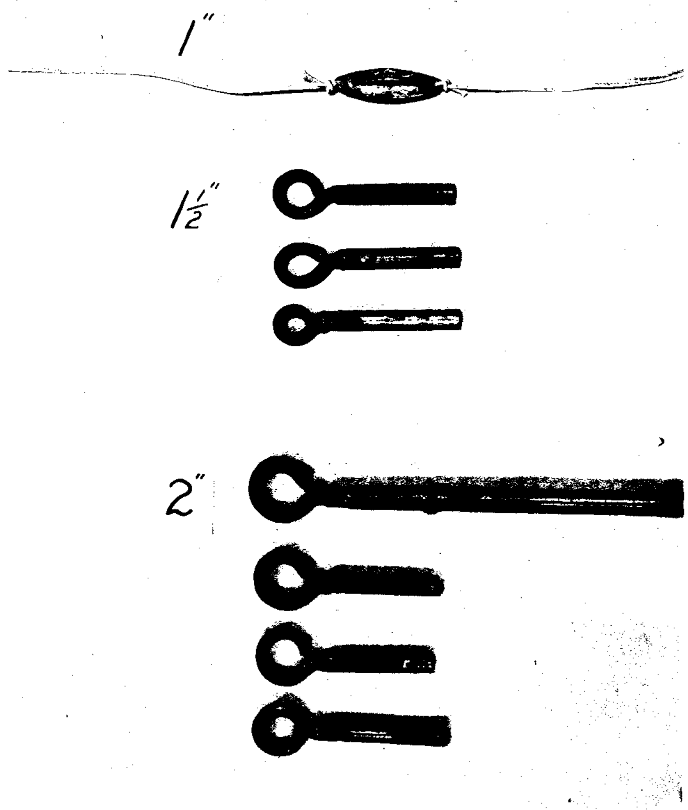


Fig. 42 Apparatus for Measuring 1-, 1-1/2-, and 2-inch Diameter Tubes

The eye bolts were made so that the maximum outside diameter of the eyes decreased in increments of 0.1 inch. The largest size for the 1-1/2-inch tube was 1.4 inches and for the 2-inch tube 1.9 inches. The 1/2-inch steel tubes as used with the caliper were utilized in making measurements with the eye bolts. Tubes were measured by turning the eye bolts on the inside of the tubes. A plug 0.9 inch in diameter with a cross section similar to a cross was made from 1/8-inch steel to measure the 1-inch diameter tubes. The longitudinal section of the plug was made in the form of a long oval to prevent catching on the sheet metal screws as it was drawn through the tube. This device shown at the top of Fig. 42 was pulled into the tube after threading through a stiff wire.

Polyethylene tubing. Polyethylene tubing for all field installations was obtained from the Carter Products Corporation, Cleveland, Ohio (presently known as the Carter Products Division of Carlon Products Corporation). The tubing known by the trade name, "Carlon E and EF", is manufactured in standard sizes shown in Table 17. Carlon EF grade is suitable for carrying liquids which are to be used for human consumption, while Carlon E is not. According to the manufacturer polyethylene tubing has been used for farm and lawn sprinklers, water lines in mines, sewage lines, corrosive gas ducts, chemical plant piping, stock watering lines, and for many other uses. Polyethylene is quite variable in physical properties as shown in Table 1.

Table 17

Specifications for Commercially Available Polyethylene Tubing*

Nominal diameter	Wall thickness	Weight per foot	Minimum bending radius without collapse	Cost per foot (1949)
inches	inches	pounds	inches	dollars
1/2	0.109	0.10	3.5	0.093
3/4	0.113	0.14	4.0	0.133
1	0.120	0.18	8.5	0.175
1-1/4	0.140	0.27	11.5	0.267
1-1/2	0.145	0.32	18.0	0.312
2	0.154	0.44	24.0	0.400
3	0.217	0.91	36.0	0.828
4	0.237	1.25	60.0	1.150
6	0.280	2.23	180.0	2.070

*Data supplied by Carter Products Corporation for "Carlon E" tubing

Specifications for "Carlon E and EF" tubing are shown in Table 17. Perforated 2-inch diameter drain tubes for the preliminary installation, to be described later, were obtained from tubes in stock; however, the 3- and 4-inch diameter tubes in the above experiment and tubes for other installations were made in special sizes. The minimum bending radius without collapse for stock-size tubes is shown in Table 17. However, tubes are shipped in much larger coils as shown in Fig. 43. Large diameter tubes are generally shipped in straight lengths. Plastic tubes for the various field installations will be described under method of procedure.

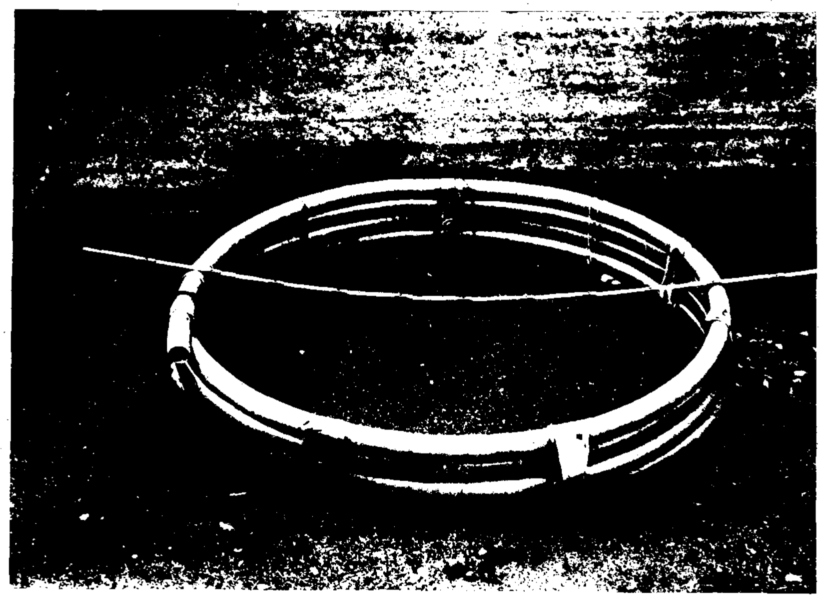


Fig. 43 Four-foot Coil of 2-Inch Polyethylene Tubing. From U. S. Corps of Engineers (53, plate 41)

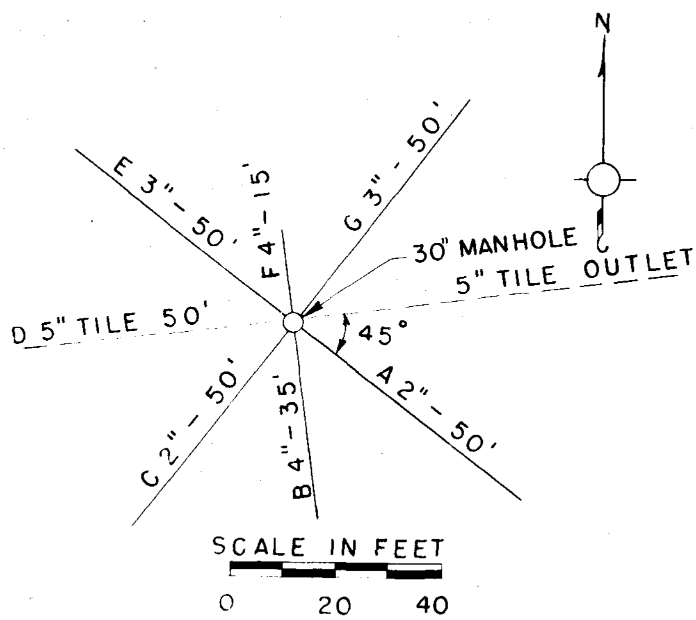


Fig. 44 Preliminary Plastic Tube Installation

Method of procedure

Field investigations consisted of a preliminary installation, a tube-diameter wall-thickness experiment, and other field studies including a 1-inch diameter drain 500 feet in length. With one exception the drain tubes were installed by attaching the tubing to the plug of the mole plow and pulling the tubes into the mole channel. Perforations were made in the tubes with an electric drill. In tubes with very thin walls the holes were frequently imperfect, but this difficulty was not encountered with thicker tubes.

Preliminary installation. In August 1948 several short lengths of 2-, 3-, and 4-inch diameter plastic tubes were installed on the Agricultural Engineering Research Farm near Ames. All tubes were put in with the Killefer mole plow except a short drain which was placed in a hand-dug ditch. This installation in Webster silt loam soil was arranged similarly to spokes of a wheel as shown in Fig. 44, all drains sloping toward the manhole at the center. A 5-inch tile drain provided an outlet for the manhole. This drain was 1/2 foot lower than the drains which empty into the manhole.

Drain tube diameter, wall thickness, number and size of perforations, and depth of the drain tubes are given in Table 18. The 2- and 3-inch diameter tubes were pulled in an opening formed by a 4-1/2-inch mole plug and the 4-inch

Table 18

Data on Preliminary Drain Tube Installation

Line	Dia.	Tube wall thickness	Size hole	No. holes per foot	Avg. depth
	inches	inches	inches		feet
A	2	5/32	1/8	48	2.62
B	4	3/32	5/32	35	2.46
C	2	5/32	1/4	36	2.54
D	5	Clay tile in 7-inch width ditch			3.32
E	3	3/32	5/32	36	2.49
F	4	3/32	5/32	35	3.14
G	3	5/32	5/32	36	2.73

diameter tube shown in Fig. 45 was pulled in behind the 6-inch mole plug. First a hole was dug for the manhole. This opening was also used as a starting place for the mole plow. The plastic tubes were connected to the mole plow as shown in Figs. 36 and 46. A 5-inch tile was installed in line D to provide a comparison of outflow with the plastic tubes. Line F consisted of a 4-inch plastic tube 15 feet in length which was installed in a hand-dug trench. Blinding of the tube was done with care so as not to crush it; however, the trench was backfilled in the normal manner.

Tube-diameter wall-thickness experiment. Three field installations consisting of tubes 1, 1-1/2, 2, 3, and 4 inches in diameter were made in Edina, Webster, and Luton soil. Each installation consisted of two 36-foot lines for each diameter or 360 feet of tubing at each location. All tubes were placed at a uniform depth of approximately 30 inches. Due to lack of suction with the 1/2-inch mole blade some of the 1-, 1-1/2-, and 2-inch diameter tubes were as shallow as 26 inches. The plastic tubing for this experiment was shipped in straight 20-foot lengths rather than coiled to avoid collapse prior to installation. As shown in Table 19 and Figs. 47 and 48 each diameter consisted of tubes with 3 wall thicknesses which were indicated as 1, 2, or 3. Each diameter was installed in two sizes of mole drains which were designated as treatment A and B. All tubes were perforated with four rows of 1/4-inch holes with

Fig. 45 Four-Inch Plastic Tube in a
6-Inch Mole Channel

Fig. 46 Starting the Installation of a
Plastic Tube

Table 19

Specifications for Tube-Diameter Wall-Thickness Installations

Inside diameter of tube	1/4" perforations per ft.	Wall thickness		Maximum dimension of mole plow point	Mole plug diameter and width of mole plow blade in inches	
		No.*	Inches		Treatment A	Treatment B
Inches				Inches		
1	12	1	0.015	1-1/2	1-1/2 (1/2)	2 (1/2)
		2	0.030			
		3	0.060			
1-1/2	12	1	0.020	1-1/2	2 (1/2)	3 (1/2)
		2	0.040			
		3	0.080			
2	12	1	0.025	1-1/2 (A) 3 (B)	2-1/2 (1/2)	3-1/2 (1)
		2	0.050			
		3	0.100**			
3	12	1	0.025	3	3-1/2 (1)	4-1/2 (1)
		2	0.050**			
		3	0.100**			
4	12	1	0.025	3	4-3/4 (1)	6 (1)
		2	0.050			
		3	0.100			

*Number refers to wall thickness given in Fig. 49.
**Black color plastic tubes. All others were white.

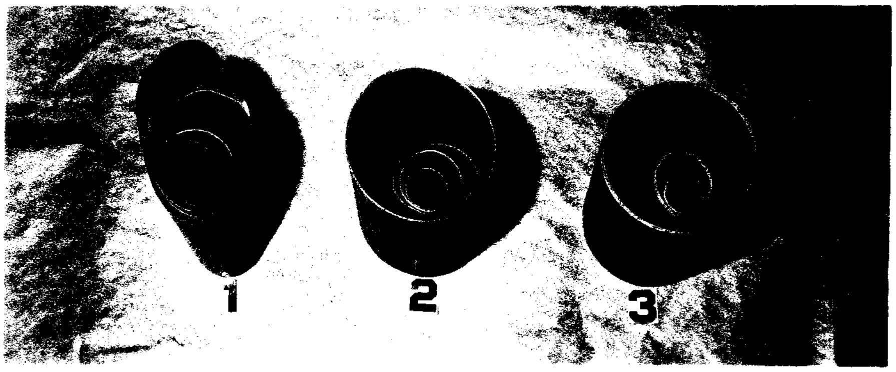


Fig. 47 Samples of Plastic Tubes Described in Table 19

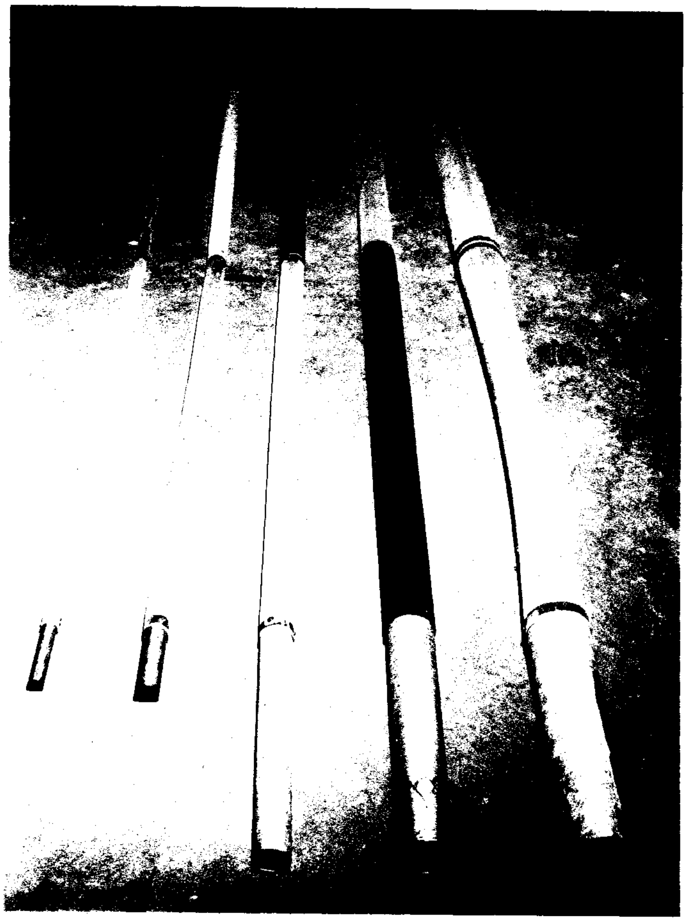


Fig. 48 Outlet Ends of Connected 4-Foot Sections of Tubing

a spacing of 4 inches making a total of 12 holes per foot. Each 36-foot line contained nine 4-foot sections of tubing or three replications of three wall thicknesses randomized within each line. This arrangement provided for statistical analysis of the data at each installation and a comparison between the three soils. The three field installations as shown in Fig. 49 were similar except for the statistical arrangement of the tube sections and tube diameters. The location, soil, date of installation, and moisture content of the soil are shown in Table 20. The soil moisture was below field capacity at the time of installation and the soil was too dry to secure good mole drains.

As shown in Figs. 48 and 50 the 4-foot sections in each line were connected by a 2-inch length of metal tubing of the proper diameter. Short sheet metal screws with small washers next to the plastic fastened the 4-foot sections together so that the tubing could be pulled into the channel with the mole plow. A length of 4 feet was arbitrarily chosen as the minimum which permitted the tube to deflect without support from the metal connecting rings. The connecting rings were comparatively rigid so that failure of one section would not cause failure of an adjacent 4-foot length.

The size of mole plugs selected for treatments A and B was chosen so that treatment A was just large enough to

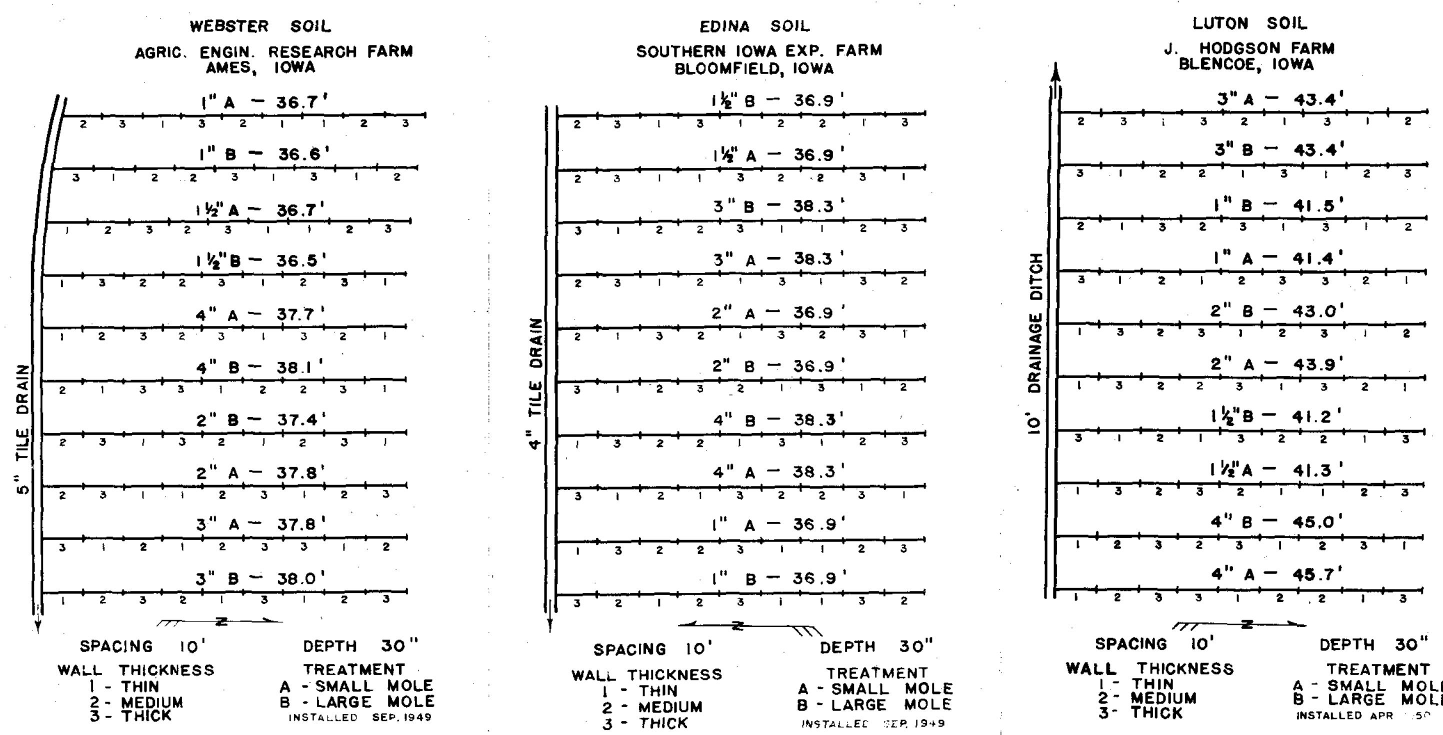


Fig. 49 Tube-Diameter Wall-Thickness Installations

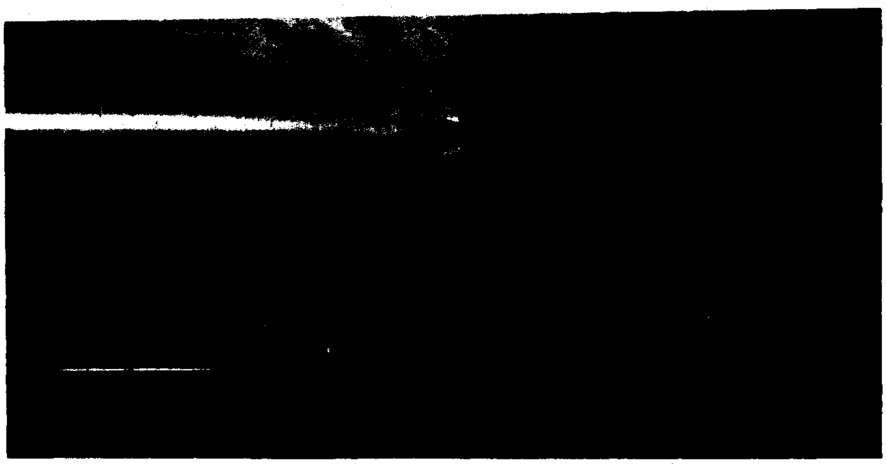


Fig. 50 Method of Connecting 4-Foot Sections

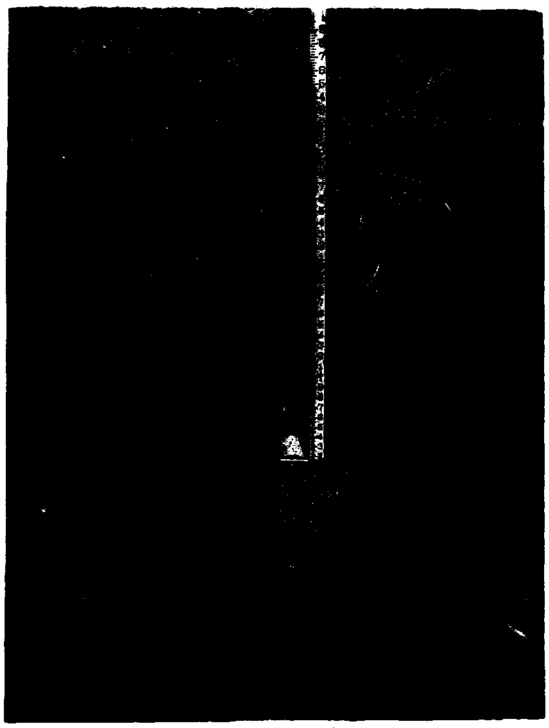


Fig. 51 Junction of Plastic Tube and Tile Drain

Table 20

Tube-Diameter Wall-Thickness Installations

Location	Soil series	Date of instal-lation	Moisture content in per cent	
			Depth 12 in.	Depth 30 in.
Agric. Engr. Res. Farm, Ames, Iowa	Webster	Sept. 1949	29.0	19.5
Southern Iowa Exp. Farm, Bloomfield, Iowa	Edina	Sept. 1949	27.8	35.2
J. Hodgson Farm, Blencoe, Iowa	Luton	Apr. 1950	35.8	30.6

permit the tube to be pulled into the hole and treatment B was chosen sufficiently large so that the tube fit loosely in the channel. The relative size of the mole plug and plastic tube for each diameter and treatment is shown in Fig. 52. The installations in Webster and Edina soil were placed so that the effluent drained into existing tile lines. Since the discharge from each line of drain tubes was assumed to be negligible, direct connections were not made. A perforated metal plate was placed over the end of the plastic tube line as shown in Fig. 51, sand and gravel being thrown in to permit rapid movement of water to the drain. Concrete slabs were placed at each end of the line so that the drains could be located with a tile probe. In Luton soil the tubes emptied into an open ditch. In this installation

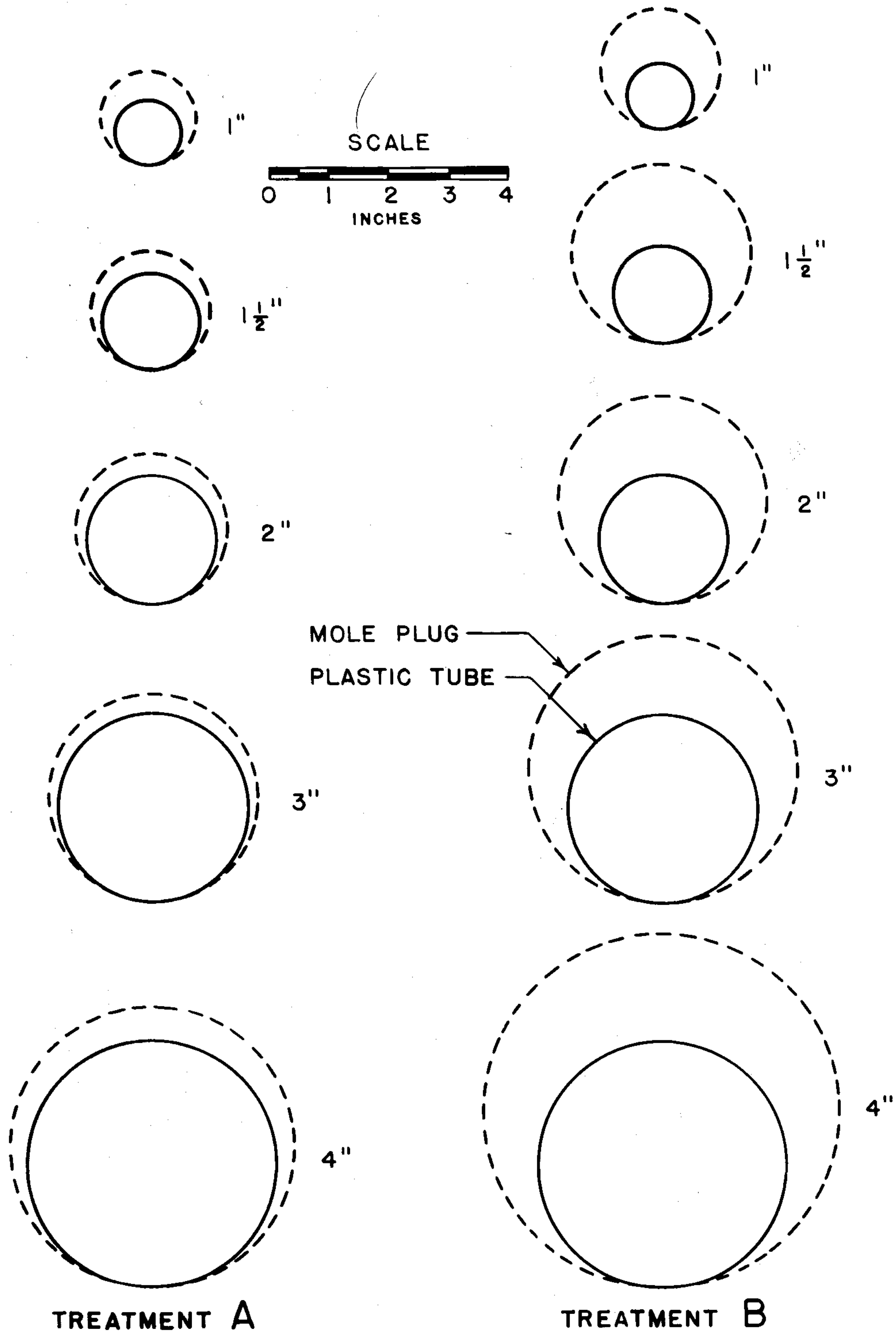


Fig. 52 Relative Size of Mole Plugs and Drain Tubes

drain spouting approximately 8 feet in length was used at the outlet end of each line.

In making the field installations the mole plow was lowered into the soil at the outlet end, either in the open ditch or in a narrow trench which was dug at the junction with the tile line. The method of attaching the mole plug to the tube is shown in Fig. 37. With the 3- and 4-inch drains lines which were pulled in using the smaller mole plug (treatment A) friction between the tube and the soil caused the tubes to break by tearing out at the sheet metal screws. This difficulty was eliminated by pouring a small quantity of water on the tube as it was being installed.

Miscellaneous installations. A few other field installations were made with plastic tubes under more typical conditions. On the Howard County Experimental Farm 500 feet of 1-inch diameter tubing was installed in July 1949. In 1946 Schwab (45) reported that mole drains were located in the same area. These drains failed a few months later. The tubing which had a wall thickness of 0.040 inch was placed at a depth of 18 inches, on a slope varying from 1 to 3 per cent, and at a distance of 80 feet from an adjoining tile drain. The tube emptied into a manhole connected to the tile drain. Four rows of 1/4-inch perforations with a spacing of 4 inches were made in the tube. The tubing was shipped in 4-foot coils and in 100-foot lengths. It was pulled in using a 1/2-inch mole blade and a 1-1/2-inch mole plug. The 100-foot lengths were connected in the field by sliding the ends into a short section of 1-1/4-inch pipe. To make these connections a hole was dug every 100 feet.

Results

The following results on the stability of polyethylene tubing in mole drains should be considered as preliminary since the drain tubes have been in the soil from 1 to less than 3 years. It is likely that they will continue to deflect for some time.

The drain tubes in the preliminary study were measured 32 months after installation. The data shown in Table 21 indicate that the 3- and 4-inch tubes have failed considerably. Line F which was placed in a hand-dug trench had a minimum diameter of 3.14 inches and stood up remarkably well considering the method of installation. Line B, the other 4-inch tube, had a minimum diameter of 3.68 inches which indicated that the mole channel provided better loading conditions than the dug trench. Considering the 3-inch tubes, line E had failed slightly less than line G which had a thicker wall. The 3-inch tubes had failed only slightly (maximum of 0.27 inch) and furnished more resistance to the load of the soil than the 4-inch tube. Lines A and C, the 2-inch tubes, were in excellent condition and had failed less than 0.1 inch, which was the least deformation which could be measured. Since the tubes were installed

Table 21

Stability of Plastic Tubes Observed
in Preliminary Installation

Line*	Nominal tube diameter	Wall thickness	Distance from outlet	Minimum inside diameter**
	inches	inches	feet	inches
A	2	5/32	0-10	1.9+
C	2	5/32	0-10	1.9+
E	3	3/32	2.0	2.80
			3.5	2.86
			6.0	2.93
			8.0	2.95
			10.0	2.95
G	3	5/32	1.0	2.73
			4.5	2.80
			6.0	2.77
			7.5	2.77
			10.5	2.75
F	4	3/32	2.5	3.63
			3.0	3.40
			6.5	3.40
			7.5	3.35
			9.5	3.14
B	4	3/32	2.5	3.91
			3.0	3.85
			6.5	3.91
			7.5	3.82
			10.5	3.68

*See Fig. 44 and Table 18

**Data taken 32 months after installation for preliminary observations, statistical analysis of the data cannot be made.

The tube-diameter wall-thickness experiment was inspected after six months and again at the end of one year in Luton soil and only after one year in Webster and Edina soils. Although the experiment was designed for analysis of the data by statistical methods, such analysis was not made because the results can be interpreted by inspection of Tables 22 to 26.

As shown in Table 22 1-inch diameter tubes had not failed in any case at the end of one year. Due to limitations in measuring exact diameters the minimum diameter has been shown as 0.9 inch. This was the width of the measuring device which was pulled through the tubes.

The 1-1/2-inch diameter tubes with wall thicknesses of 0.040 and 0.080 inch did not fail more than 0.1 inch as shown in Table 23. Tubes with 0.080 wall thickness had a minimum diameter of 1.3 inches at time of installation. This tube was oval in cross section when received from the manufacturer. For a wall thickness of 0.020 the tubes had failed slightly in some cases, but this may have been caused by failure during installation by soil rolling along the tube. There was little noticeable difference due to size of mole channel. The smaller mole channel was slightly better.

Table 22

Minimum Diameter of 1-Inch Tubes One Year after Installation

Wall thickness	Replication	*Minimum diameter in inches	
		Treatment A	Treatment B
inches			
	Webster Soil		
0.015	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.030	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.060	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
	Edina Soil		
0.015	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.030	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.060	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
	Luton Soil		
0.015	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.030	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9
0.060	1	0.9	0.9
	2	0.9	0.9
	3	0.9	0.9

*Measurement greater than 0.9 inch could not be made because of tube connecting rings

Table 23

Minimum Diameter of 1-1/2-Inch Tubes One Year after Installation

Wall thickness	Replication	Minimum diameter in inches: Treatment A	Treatment B
inches			
	Webster Soil		
0.020	1	1.4	1.3
	2	1.3	1.2
	3	1.4	1.2
0.040	1	1.4	1.4
	2	1.4	1.4
	3	1.4	1.4
0.080	1	1.3	1.3
	2	1.3	1.3
	3	1.3	1.3
	Edina Soil		
0.020	1	1.4	1.4
	2	1.4	1.3
	3	1.4	1.3
0.040	1	1.4	1.4
	2	1.4	1.4
	3	1.4	1.4
0.080	1	1.3	1.3
	2	1.3	1.3
	3	1.3	1.3
	Luton Soil		
0.020	1	1.3	1.3
	2	1.4	1.3
	3	1.4	1.3
0.040	1	1.4	1.4
	2	1.4	1.4
	3	1.4	1.4
0.080	1	1.3	1.3
	2	1.3	1.3
	3	1.3	1.3

Table 24

Minimum Diameter of 2-Inch Tubes One Year after Installation

Wall thickness	Replidation	Minimum diameter in inches	
		Treatment A	Treatment B
inches			
	Webster Soil		
0.025	1	1.9	1.8
	2	1.7	1.8
	3	1.9	1.9
0.050	1	1.9	1.9
	2	1.8	1.8
	3	1.9	1.9
0.100	1	1.9	1.9
	2	1.9	1.9
	3	1.9	1.9
	Edina Soil		
0.025	1	1.9	1.9
	2	1.9	1.8
	3	1.9	1.9
0.050	1	1.9	1.9
	2	1.9	1.9
	3	1.9	1.9
0.100	1	1.9	1.9
	2	1.9	1.9
	3	1.9	1.9
	Luton Soil		
0.025	1	1.9	1.7
	2	1.9	1.7
	3	1.8	1.8
0.050	1	1.9	1.9
	2	1.9	1.9
	3	1.9	1.8
0.100	1	1.8	1.9
	2	1.9	1.9
	3	1.9	1.9

Table 25

Minimum Diameter of 3-Inch Tubes One Year after Installation

Wall thickness	Replication	Minimum diameter in inches	
		Treatment A	Treatment B
inches			
	Webster Soil		
0.025	1	2.4	2.6
	2	2.3	2.6
	3	failed	2.2
0.050	1	2.6	2.7
	2	2.8	2.7
	3	2.4	2.4
0.100	1	2.8	2.8
	2	2.9	2.8
	3	2.9	2.8
	Edina Soil		
0.025	1	2.0	2.7
	2	2.3	2.5
	3	2.0	2.5
0.050	1	2.4	2.7
	2	2.3	2.7
	3	2.4	2.7
0.100	1	2.7	3.0
	2	2.6	2.9
	3	2.7	2.9
	Luton Soil		
0.025	1	2.2	2.2
	2	2.2	2.6
	3	2.2	2.4
0.050	1	2.7	2.6
	2	2.6	2.5
	3	2.7	2.9
0.100	1	2.9	2.9
	2	2.8	2.9
	3	2.9	2.9

Table 26

Minimum Diameter of 4-Inch Tubes One Year after Installation

Wall thickness	Replication	Minimum diameter in inches	
		Treatment A	Treatment B
inches			
	Webster Soil		
0.025	1	2.8	2.8
	2	2.8	2.4
	3	2.5	2.3
0.050	1	3.4	3.8
	2	3.3	3.8
	3	3.0	3.3
0.100	1	3.6	3.8
	2	3.6	3.8
	3	3.5	3.9
	Edina Soil		
0.025	1	3.6	3.4
	2	3.6	2.3
	3	3.6	3.6
0.050	1	3.4	3.9
	2	3.6	3.8
	3	3.8	3.6
0.100	1	3.5	3.8
	2	3.8	3.9
	3	3.8	3.9
	Luton Soil		
0.025	1	failed	failed
	2	2.8	failed
	3	3.0	failed
0.050	1	3.0	3.8
	2	3.2	2.6
	3	3.0	3.0
0.100	1	3.4	3.6
	2	3.5	3.5
	3	3.4	3.0

The 2-inch diameter tubes with wall thicknesses of 0.050 and 0.100 inch did not fail to any appreciable degree. Four of the sections were 1.7 inches in diameter, 9 were 1.8 inches, and the remaining 41 were 1.9 inches. There was greater failure with the 0.025 wall thickness than with the heavier tubes. There was little difference, if any, due to treatments. In general these tubes were in good condition.

While 3-inch tubes in most cases had failed considerably, those with 0.100 wall thickness were in good condition except for treatment A in Edina soil. Tubes with 0.025 wall thickness failed more than those with 0.050 wall thickness in most cases. One of the sections in Webster soil with treatment A failed as shown by the drawings on the left-hand side of Fig. 53. These cross sections were obtained by making casts of portions of the tube. This failure probably was not due to soil load, but a failure during installation resulting from soil being rolled along the tube. Results as to the effect of size of mole channel were not consistent.

The data for 4-inch diameter tubes show quite consistently that the degree of failure increased as the wall thickness decreased. For both treatments and all wall thicknesses the order of increasing failure was Edina, Webster, and Luton soil. In Luton soil four of the six tubes with 0.025 wall thickness had failed completely after

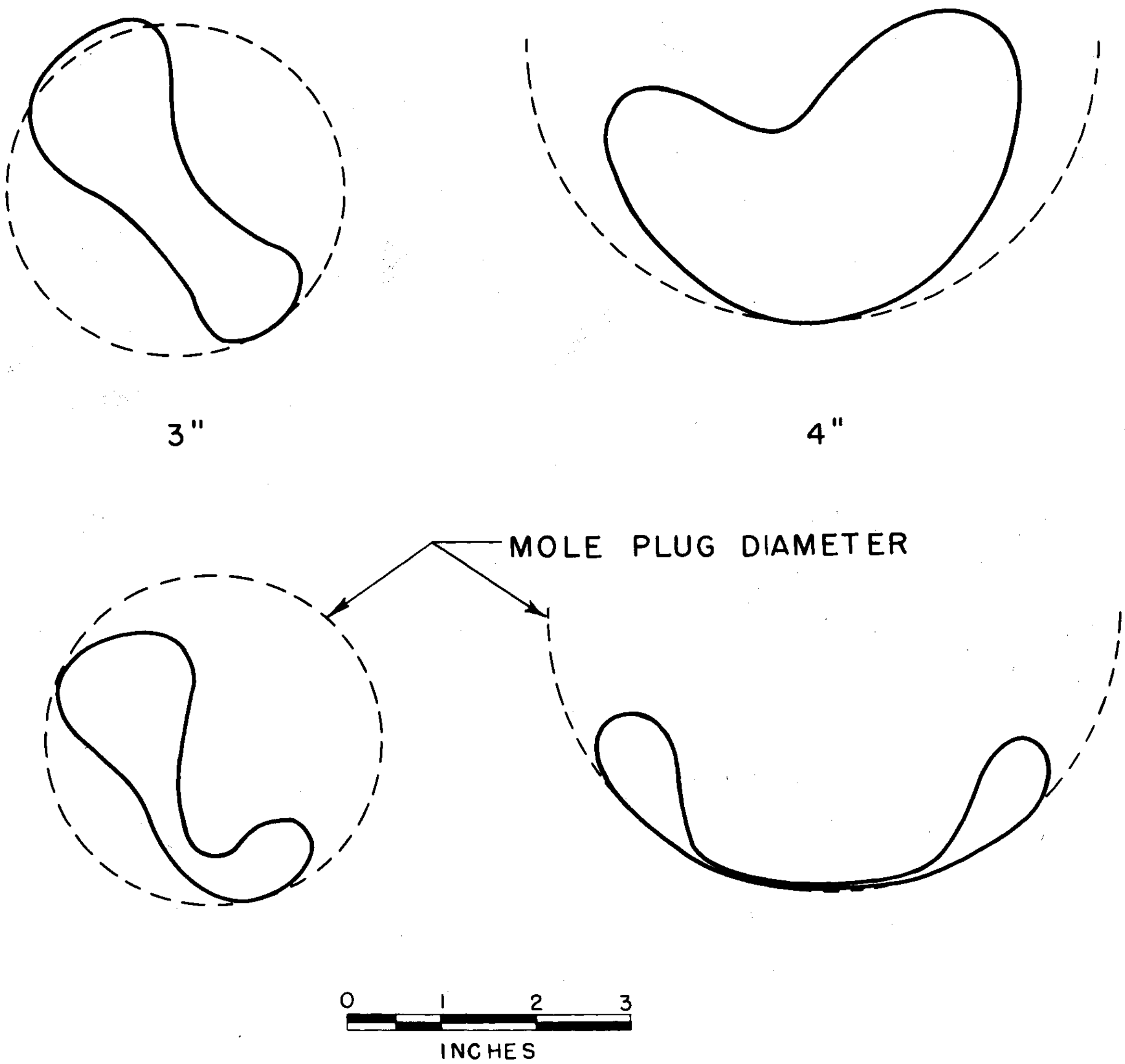


Fig. 53 Observed Failures of 3- and 4-Inch Plastic Drain Tubes

six months. The failure on these tubes as shown by the drawings on the right-hand side of Fig. 53 was probably due to soil load. While in most cases except for 0.025 wall thickness the tubes were more stable in the larger mole channel than in the smaller, the differences are small and no conclusions should be drawn without further observations.

Drain tubes 2 inches or less in diameter appeared to be the most practical considering stability and cost. Although the data in Tables 21, 25, and 26 indicated that the thicker-walled tubing was relatively stable for 3- and 4-inch diameter drains, the tubing cost was too high to justify their use. The stability of the thinnest-walled tubes was not sufficient to make their use practical in the field. Tubes must have sufficient stability to maintain their shape when pulled into the mole channel or failure will occur during installation.

The 1-inch drain 500 feet in length on the Howard County Experimental Farm operated satisfactorily. Outflow measurements taken in the spring of 1951 were approximately equal to the outflow from 5-inch tile drains 420 feet in length which were located in another area on the farm.

Cost of Plastic Tube Drainage

The cost of 1-, 1-1/2-, and 2-inch diameter plastic tubes was estimated for tubes with wall thicknesses as indicated in Table 19 and a comparison was made with tile drainage on the basis of cost per 1000 feet and cost per acre. Larger diameter tubes were not considered because of excessive cost and failure of several 3- and 4-inch diameter tubes as indicated in previous investigations.

The capacity in cubic feet per second for these tubes as shown in Table 27 was computed for slopes varying from 0.1 to 2.5 per cent by using Manning's formula with a roughness coefficient of 0.012. This value was also used by the U. S. Corps Engineers (53). Present methods for the design of tile drains were used in this analysis. For drain tubes spaced at 50 and 100 feet the length which produces a flow equal to the capacity of the tube at a given slope has been computed and is shown in Table 27 and Fig. 54. For 1-inch tubes the maximum length for 50-foot spacing is 400 feet at a slope of 2 per cent and one-half this length or 200 feet for 100-foot spacing. Fig. 54 indicates that for 1-inch tubes the length of the drains must be relatively short, even for slopes up to 2.5 per cent. For a given slope the 1-1/2-inch tubes will drain an area approximately three times as large as the 1-inch tubes. The maximum lengths of 1-1/2-inch drains are more practical and are within a range which is desirable from the standpoint of layout and design. The 2-inch tubes will drain an area more than twice as large as the 1-1/2-inch tubes and about six times as large as the 1-inch tubes. Except for steep slopes 1-inch drains appear to have

Table 27

Maximum Length of Plastic Tubes at Various Slopes

Slope	1-inch dia. tubes			1-1/2-inch dia. tubes			2-inch dia. tubes		
		Maximum length for spacings of**			Maximum length for spacings of**			Maximum length for spacings of**	
	Q*	50 ft.	100 ft.	Q*	50 ft.	100 ft.	Q*	50 ft.	100 ft.
%	cfs	feet	feet	cfs	feet	feet	cfs	feet	feet
0.1	0.00162	90	45	0.00475	263	131	0.01024	566	283
0.2	0.00229	127	63	0.00672	372	186	0.01449	801	401
0.4	0.00324	179	90	0.00951	526	263	0.02050	1,134	567
0.6	0.00398	220	110	0.01167	645	323	0.02514	1,390	695
1.0	0.00513	284	142	0.01504	832	416	0.03242	1,793	896
1.5	0.00628	347	174	0.01843	1,019	510	0.03971	2,196	1,098
2.0	0.00725	401	200	0.02127	1,176	588	0.04584	2,535	1,267
2.5	0.00811	448	224	0.02378	1,315	657	0.05126	2,835	1,417

*Discharge computed by Manning's formula with n = 0.012

**Length of drain tube based on a drainage coefficient of 3/8 inch per 24 hours

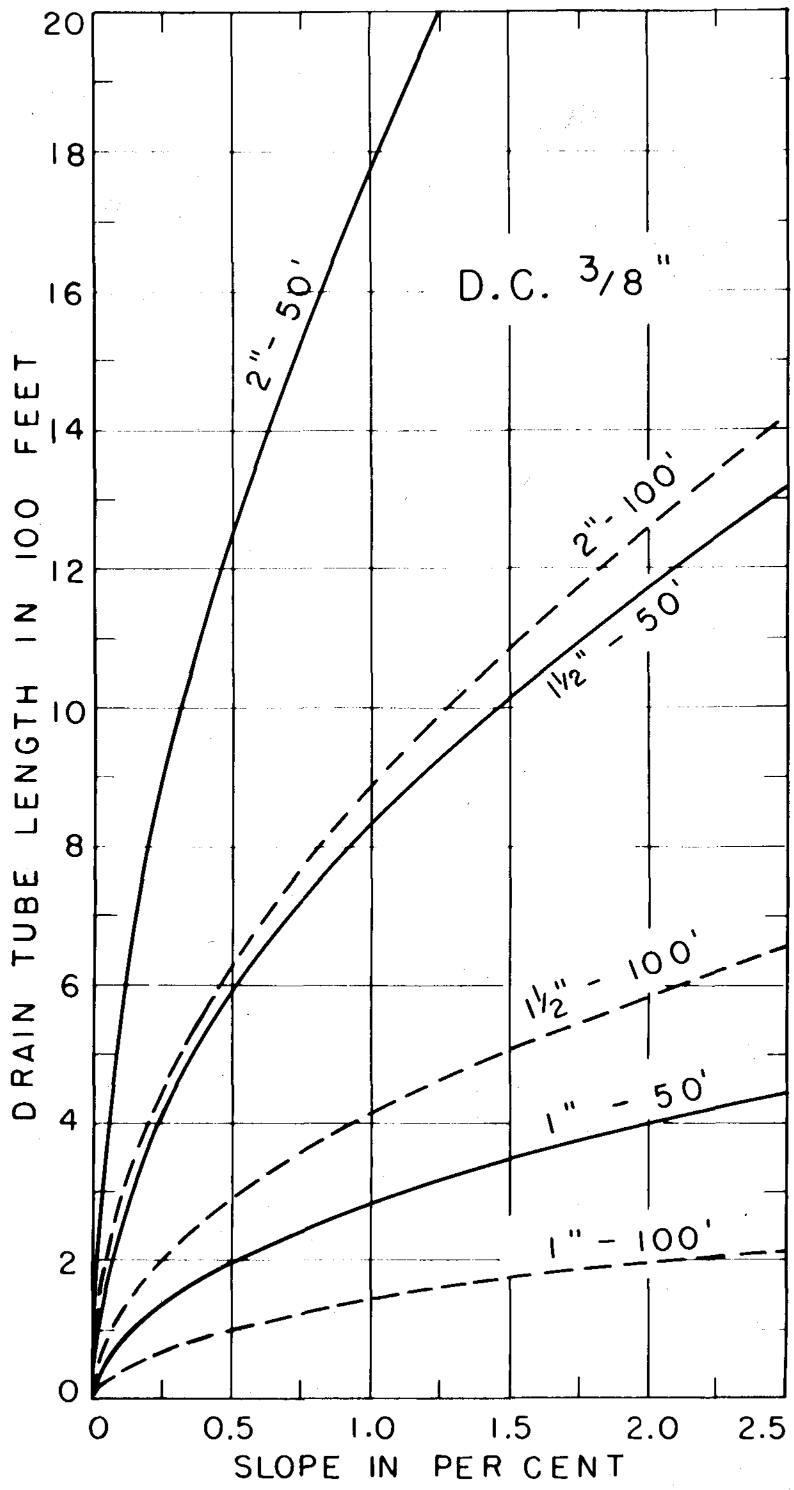


Fig. 54 Maximum Length of Drain Tubes for Spacings of 50 and 100 Feet

inadequate capacity unless the drains are spaced close together or the length is kept short. For most laterals at spacings of 50 or 100 feet 1-1/2-inch diameter tubes or larger appear to be more practical than 1-inch tubes. From Fig. 54 the maximum length for laterals at spacings from 50 to 100 feet can be obtained by interpolation between the curves.

The estimated cost of plastic tube and tile drains was made for depths of 2, 3, and 4 feet. Cost estimates were based on 1950 prices. The estimated cost of installation for plastic tube drains is shown in Table 28. Tractor cost was determined by using mole plow drafts of 8,000, 17,000, and 28,000 pounds for depths of installation of 2, 3, and 4 feet, respectively. Based on maximum draft data as given by the Nebraska tractor tests International Harvester TD 9 (test #344), Caterpillar D 4 (test #417), and Allis Chalmers HD 7 W (test #360) are large enough for moling at a depth of 2 feet; International Harvester TD 18 A (test #446), Caterpillar D 7 (test #358), and Allis Chalmers HD 10 W (test #361) are satisfactory for a depth of 3 feet; and International Harvester TD 24 (test #447), Caterpillar D 8 (test #415), and Allis Chalmers HD 19 (test #397) are satisfactory for a depth of 4 feet. As indicated in Table 28, the cost estimate was made for drains 500 feet in length with the assumption that continuous drains of this length could be installed from one outlet. Time

Table 28

Estimated Installation Cost for Plastic Drain Tubes

Depth 2 feet

Tractor cost - 23.4 minutes at $7.50 per hour* (allowing 67% for lost time)	$ 2.93
Mole plow cost - at $0.45 per hour*	0.18
Moving cost - at $25 per move (allowing 1 move per 10 hours use)	0.98
Labor cost - 3 men at $1.00 per hour	1.17
Total Cost per 500-foot drain	$ 5.26
Total Cost per 1000-feet (two 500-foot drains)	10.52

Depth 3 feet

Tractor cost - 26.0 minutes at $12.50 per hour* (allowing 70% for lost time)	$ 5.41
Mole plow cost - at $0.50 per hour	0.22
Moving cost - at $30 per move (allowing 1 move per 10 hours use)	1.30
Labor cost - 4 men at $1.00 per hour	1.73
Total Cost per 500-foot drain	$ 8.66
Total Cost per 1000 feet (two 500-foot drains)	17.32

Depth 4 feet

Tractor cost - 39.0 minutes at $15.00 per hour* (allowing 80% for lost time)	$ 9.75
Mole plow cost - at $0.60 per hour	0.39
Moving cost - at $35 per move (allowing 1 move per 10 hours use)	2.28
Labor cost - 5 men at $1.00 per hour	3.25
Total Cost per 500-foot drain	$15.67
Total Cost per 1000 feet (two 500-foot drains)	31.34

*Rate of installation at 7.8 minutes per 500-foot drain and mole plow cost based on data obtained for mole drains by Schwab (45).

of installation was based on data obtained by Schwab (45) for the installation of mole drains. Allowance was made for digging the hole from which the tubes are to be installed, for connecting the tube, and for other interruptions by assuming a relatively large time loss varying from 67 to 80 per cent. Mole plow cost was also based on data obtained by Schwab (45) for a depth of 2 feet while at depths of 3 and 4 feet the cost was estimated. For all depths cost of moving equipment was based on a flat rate per move, and it was assumed that one move was required for every 10 hours of operation. The cost for each move was increased as the depth of installation increased to allow for greater cost of handling larger equipment. Labor cost was assumed at a uniform rate of $1.00 per hour. Labor was required for digging the hole from which to start the installation, for handling the tubing and for other miscellaneous jobs. An additional man was added for each 1 foot increase in depth of installation. The tractor operator was included in the power cost, but not in the labor cost. The total cost of installation per 500-foot drain is shown in Table 28. The cost of installation would be greater for drains shorter than 500 feet and perhaps less for longer drains providing such lengths could be installed at one time. From Table 28 the installation cost for 4-foot depth drains is three times that for 2-foot drains, while the cost for 3-foot drains is less than twice that for 2-foot drains.

Table 29

Estimated Cost for Plastic Tube and Tile Drains

Diameter	Wall thick-ness	Material* cost per 1000 ft.	Installation cost per 1000 ft.	Total cost per 1000 ft.	Total cost per** acre
inches	inches	dollars	dollars	dollars	
Depth of Installation 2 Feet					
1	0.015	20	11	31	14
1	0.030	40	11	51	22
1	0.060	80	11	91	40
1-1/2	0.020	40	11	51	22
1-1/2	0.040	80	11	91	40
1-1/2	0.080	158	11	169	74
2	0.025	67	11	78	34
2	0.050	132	11	143	62
2	0.100	264	11	275	120
Depth of Installation 3 Feet					
1	0.015	20	17	37	16
1	0.030	40	17	57	25
1	0.060	80	17	97	42
1-1/2	0.020	40	17	57	25
1-1/2	0.040	80	17	97	42
1-1/2	0.080	158	17	175	76
2	0.025	67	17	84	37
2	0.050	132	17	149	65
2	0.100	264	17	281	122
5	Tile	103	90	193	84
Depth of Installation 4 Feet					
1	0.015	20	31	51	22
1	0.030	40	31	71	31
1	0.060	80	31	111	48
1-1/2	0.020	40	31	71	31
1-1/2	0.040	80	31	111	48
1-1/2	0.080	158	31	189	82
2	0.025	67	31	98	43
2	0.050	132	31	163	71
2	0.100	264	31	295	129
5	Tile	103	120	223	97

*Plastic tubing based on a cost of 3.5 cents per cu. in.
**Cost per acre based on a spacing of 100 feet.
Note: Cost estimated to the nearest dollar and does not include design, layout, and main.

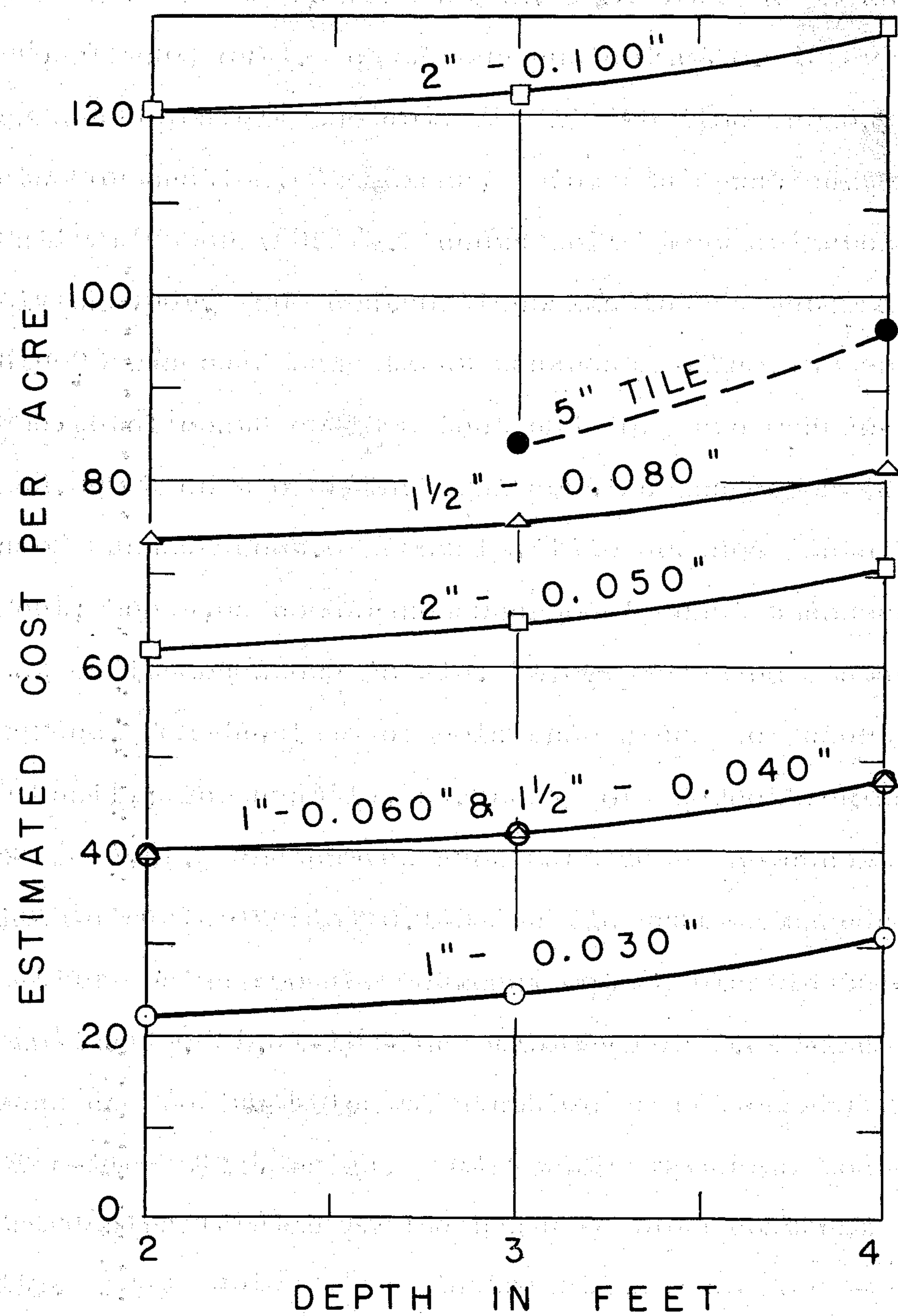


Fig. 55 Estimated Cost of Plastic Tube and Tile Drains in Dollars for a Spacing of 100 Feet

The total cost for plastic tube drains and tile drains is shown in Table 29. Three diameters of tubing and three wall thicknesses of each are shown. The cost of plastic tubing was based on 3.5 cents per cubic inch of material and was approximately the cost supplied by the Carlon Products Corporation, Cleveland, Ohio. Thin-walled tubing costs slightly more than 3.5 cents while heavier tubes are slightly cheaper. The cost of installation as determined in Table 28 was used only to the nearest dollar in Table 29. The total cost per 1000 feet and per acre was determined. The cost of 5-inch tile drains was computed at depths of 3 and 4 feet. Normally, tile are not installed at 2 feet; hence, a comparison was not made at this depth. All cost estimates shown in Fig. 55 are based on a spacing of 100 feet. The cost at other spacings can be determined rather easily; for example, a spacing of 50 feet would double the cost. The curves show that with the exception of 2-inch diameter tubing 0.100 inch in wall thickness, 5-inch tile drains are more expensive than plastic tubes. As shown in Fig. 55, 1-1/2-inch tubing with 0.040-inch wall thickness can be installed at a cost of about one-half that of 5-inch tile drains. This tubing appeared to be the most desirable size on the basis of both cost and capacity.

DISCUSSION

From the standpoint of stability, cost, ease of shipment, and handling, 1-, 1-1/2-, and 2-inch diameter plastic tubes appear to be more practical than larger diameters. The disadvantages of using a small-size tube are that the drain has a limited capacity, must be installed more accurately, and is more likely to become clogged. An advantage of plastic tube drains is that the cost is relatively low permitting close spacing of drains when necessary or making possible the drainage of land with considerable slope. Portions of the drain tubes need not be perforated if there is danger from blowouts. By placing these small-size tubes on relatively steep slopes at spacings about one-half that for tile drains and by keeping the laterals short, the capacity will not be exceeded.

Since large-size plastic tubes are neither economical nor practical, tile drains are required for mains and submains. The drainage system should be installed so that the plastic tube drains are on slopes greater than 0.5 per cent. If further investigations show that small thin-wall plastic tubes have sufficient stability, their use as laterals in a subsurface drainage system appears to be practical providing slopes are adequate. If concrete is used at each junction, there should be no difficulty in making a satisfactory connection.

All phases of the investigation were not carried to completion; therefore, final results could not be included. The studies regarding the effect of perforations on flow into subsurface drains in saturated soil were completed to the satisfaction of the author. Since the theoretical analysis was in excellent agreement with the values obtained with the electric analogue, the data were verified for a limited number of conditions. For practical purposes Q/Q_o and Q are maximum values and may be taken as correct since soil conditions surrounding a perforation are difficult to determine. The results could be verified by field experiments, but this is difficult because soil permeability and boundary conditions are hard to evaluate and a completely saturated soil seldom occurs in the field. The effect of perforations on flow in partially saturated soil was not evaluated.

Results from the study on the effect of deviations from true grade on the performance of small drain tubes were obtained for laboratory conditions and the relationship to field conditions is uncertain. Since sand and soil were utilized in a disturbed state, the outflow is not representative of drains in the field. The author believes that a satisfactory answer to this problem has not been obtained and application of the data in the field should be made with caution.

The field investigations to determine the stability of various plastic tubes in the soil are long-time experiments and for that reason only preliminary results can be included. Since these drain tubes have been in the soil from 1 to about 3 years, it is difficult to determine whether or not maximum soil loads have yet been transmitted to the tubes. Even though the tubes may be supporting the full load, it is possible that deflection will continue for many years. Therefore, the results should be considered as preliminary. In the tube-diameter wall-thickness installations the metal connecting rings between the 4-foot tube sections helped to maintain the shape of the tube during installation so the final results may not represent true field conditions. For a given wall thickness small-size tubes are relatively more stable; therefore, it is believed that this is not important in tubes 2 inches or less in diameter. Horizontal failure was observed in some of the thin-walled tubes and this was thought to be caused by soil rolling along the tube as it was pulled into the mole channel.

Cost estimates for plastic tube drainage are considered reasonably accurate for depths of installation of 2 and 3 feet. These data were based on the installation cost for mole drains. At a depth of 4 feet the cost estimate was made without the benefit of field experience or related data. Although considerable time loss was allowed in the calculations, field experience may prove the assumptions to be in error, in which case the total cost would be affected proportionally. The cost estimates were considered to be conservative.

It was not possible to study all problems involved in the use of plastic tubes for drainage purposes. For the benefit of those who wish to carry out further work the following suggestions are presented:

1. The effect of the shape of the mole channel on the stability of plastic tubes should be studied.

2. A mole plow with a device to maintain the mole drain to grade is needed so that with small drain tubes the variation from true grade will be small. Such a sighting device may be patterned after the one developed by Sack (41). Sighting devices presently used on tile ditching machines may also be suitable. Preferably, the mole plow should be operated hydraulically so that the depth of the mole channel can be easily controlled.

3. Methods of crating, transporting, and handling plastic tubes should be studied.

4. Further information on methods of installing plastic tubes is needed. This may include pulling the tubes into the drain from the outlet or placing the tubing in the ground as with a cable-laying machine. The use of plastic strips to form the drain in a manner described by Sack (41) for metal tubes should be investigated.

5. Studies should be conducted to determine the

durability of polyethylene in the soil. Oxidation and brittleness of the tubing with age and the effects of rodents and other animals should be investigated. However, some information may be obtained from further observations on present experiments.

6. Method of perforating tubing should be studied to determine an economical procedure. This is more of a problem for the manufacturer.

7. In the use of thin-walled tubes there is danger of collapse during installation. Methods of eliminating this danger or of detecting such failures at the time of installation should be studied.

8. Flow of water in perforated plastic tube drains should be studied to determine a suitable value for $\underline{n}$, the roughness coefficient in Manning's formula.

CONCLUSIONS

The following conclusions from 1 through 7 are based on the study of the effect of perforations on flow into drain tubes. These conclusions are valid for soils of uniform permeability, water-saturated to the surface, and with or without surface water. When reference is made to a drain tube embedded in gravel, it is assumed that the gravel layer is thin; that is, the effective increase in drain size due to gravel is negligible and the flow into such a gravel-embedded drain tube is the same as that into a completely porous drain.

1. Flow into perforated drain tubes in soils was much less than the flow into a completely porous drain. The reduction in flow varies considerably depending upon the diameter and depth of the drain as well as the number and size of perforations. The effect of perforations expressed as a ratio of the flow into perforated tubes to that in porous drains ranges from about 0.6 to less than 0.1.

2. For 4 holes per foot or less the flow was roughly proportional to the number of holes per foot. Above 4 holes per foot the increase was rapid up to about 10 holes per foot and then increased at a decreasing rate for greater numbers of perforations.

3. Doubling the diameter of perforations (cross-sectional area quadrupled) did not double the flow, but did result in appreciable increase. The increase in flow was greater at a depth of 8 feet than at shallower depths.

4. The flow into perforated drain tubes was considerably larger at greater depths than at shallow depths.

5. Other quantities being equal, large-size perforated tubes produced more flow than small drains. However, the flow when expressed as a percentage of the flow into gravel-embedded tubes was less for large than for small drains.

6. The inflow into 5-inch tile drains at a depth of 4 feet for crack spacings of 1/8, 1/4, and 1/2 inch corresponded to the flow into perforated drain tubes with 12, 17, and 31 1/2-inch holes per foot, respectively. The assumption was made that the outside diameter of the drain tile was 6 inches and that 8 rows of perforations were used in the tubing. The results are about the same for different numbers of longitudinal rows of holes.

7. A drain tube with a single row of perforations on top yielded more flow than when the perforations were on the bottom. The effect was more pronounced as the number and size of holes per foot were increased.

Conclusions from 8 to 13, inclusive, are based on the laboratory study of the effect of deviations from true grade on the performance of 1-inch drain tubes.

8. The back pressure or the head required to move

water through drain tubes increased as the number of deviations from true grade increased and as the deviations became larger. This effect was greater in saturated than in nonsaturated soils. The latter more nearly represents field conditions. Increasing the amount of deviation from 1 to 1-1/2 inches caused a greater increase in head than doubling the number of 1-inch deviations.

9. Discharge from drain tubes was influenced by the percentage of soil aggregates greater than 0.25 mm.; however, only one set of data was significant near the 10 per cent level.

10. The head required to move water through curved tubes without perforations increased as the volume of air in the tube increased. This effect was greater as the number of deviations from true grade increased and as the deviations became larger.

11. Under saturated soil conditions with perforated drain tubes the head was proportional to the discharge.

12. The head required to move water through perforated drain tubes (a high head indicates a high water table) decreased as the slope in the drain increased. Slopes of approximately 2 to 5 per cent were necessary in order to make the head negligible. The laboratory results are believed to represent more severe conditions than would be found in the field.

13. Perforated drain tubes which had deviations from

true grade and were placed in soil which was not completely saturated performed very similarly to tubes without perforations.

Conclusions 14 to 19 inclusive are based on the study of the stability of perforated flexible tubes in mole drains.

14. The stability of perforated flexible tubes in mole drains decreased as the diameter of the tubes increased and as the wall thickness decreased.

15. Perforated flexible tubes are less stable in Luton than in Webster and Edina soil. Preliminary results indicate little if any difference between Webster and Edina.

16. Polyethylene tubing after 32 months in soil showed no visible evidence of deterioration.

17. Drain tubes 1-1/2 or 2 inches in diameter are the most practical sizes considering stability, capacity, and cost. A minimum slope of 0.5 per cent is suggested.

18. Four-inch diameter polyethylene tubing was more stable in a 6-inch mole channel than in a hand-dug trench.

19. Drain tubing must have sufficient stability to maintain its shape when pulled into the mole channel or failure will occur during installation.

20. At depths of 3 and 4 feet the installed cost of 1-1/2-inch tube drains with 0.040-inch wall thickness was approximately one-half the cost of 5-inch tile drains, and the cost for 2-inch tubing 0.050 inch thick was about three-fourths the cost for tile.

SUMMARY

The purpose of this investigation was to find a method of subsurface drainage which was more economical than tile drainage. There are many areas in Iowa and other states where the high cost of tile drainage makes this practice questionable from an economical standpoint. Mole drainage, which is more economical than tile drainage, has been studied in Iowa and other states. It has not been very successful in the north central states and at best is considered a temporary practice. Methods of stabilizing mole channels with metal tubes and by extruding a porous concrete pipe have been investigated in Germany.

In 1947 it was learned that thermoplastic (polyethylene) tubing was being produced commercially and that this tubing possessed physical and chemical properties which indicated that it would be suitable for mole drainage stabilization, as well as economical in cost. Except for a limited study of this method by the New England Division of the U. S. Corps of Engineers in 1946, no previous investigations have been made. Stabilizing mole drains with flexible plastic tubes is a broad problem and because of this only certain aspects were considered.

The effect of the number and size of perforations on the flow into subsurface drain tubes was determined by

theoretical analysis and verified by spot-checking with an electric analogue. Theoretical calculations were made for tube diameters of 2, 4, 6, and 12 inches; for 2, 4, and 8 rows of holes not to exceed 50 perforations per foot; for 1/4- and 1/2-inch diameter perforations; and for depths of 1, 2, 4, and 8 feet in all possible combinations. The flow into perforated tubes was 62 per cent or less of the flow into a completely porous drain depending on the particular combination considered. The effect of the perforations was expressed as the ratio of the flow into perforated tubes to the flow into completely porous drain tubes. The inflow into 5-inch tile drains at various crack spacings was compared to the flow into perforated drain tubes. Greater flow was obtained by placing a row of perforations on top of the tube than on the bottom.

The effect of deviations from true grade on the performance of small perforated drain tubes was studied in the laboratory. One-inch copper tubes 10 feet in length having the desired curvature were placed in a wooden tank which was filled with sand or soil. At the upper end of the tank the head required to move water through the drain tubes was measured for slopes up to 4 per cent. This head represented the height of the water table; that is, a high head indicated lack of drainage. The head increased with greater numbers of deviations from true grade and with greater amounts of deviations. The head increased with larger amounts of entrapped air and as the slope in the tubes decreased. With tubes larger than 1 inch in diameter under field conditions the effect of a few small deviations from true grade was not considered serious from the standpoint of reduced drainage, but may be undesirable because of silt accumulation.

Field investigations were conducted on the stability of perforated polyethylene drain tubes in mole drains. Tubing from 1 to 4 inches in diameter was used with wall thicknesses varying from 0.015 to 0.156 inch. Drains were installed in several sizes of mole channels and in different soil areas of Iowa. Since the tubes had been in the soil from 1 to less than 3 years, the results were not conclusive. The stability of perforated flexible tubes in mole drains decreased as the wall thickness was reduced. Polyethylene in soil showed no visible evidence of deterioration after 32 months.

A comparison was made between the cost of tile drains and plastic tube drains for depths of 3 and 4 feet. Considering stability, capacity, and cost, drain tubes 1-1/2 and 2 inches in diameter were considered most practical. Drain tubes 1-1/2 inches in diameter with 0.040-inch wall thickness can be installed for about one-half the cost for 5-inch tile drains. Small plastic tube drains appeared to be most applicable in the drainage of hillside seeps or other wet areas on sloping land and in soils with impermeable subsoils which require drains at narrow spacing. In general tile drains would be required for mains and submains while plastic tubes would be suitable for laterals.

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